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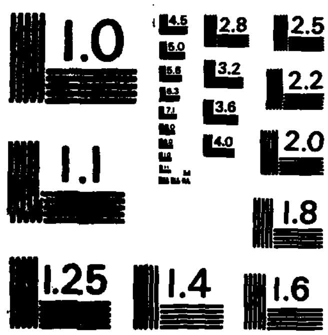

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ENGINEERING DESIGN AND DEVELOPMENT

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August 18, 1976

Dr. Shun C. Ling
Chesapeake Division (FPO-1)
Naval Facilities Engineering Command
Washington Navy Yard, Bldg. 200
Washington, D. C. 20374

Dear Dr. Ling:

During our meeting with you in Washington on August 10-11, Ed Verner agreed to send you a copy of the draft of our report, "ACMR East Coast Ocean Structures, Review of Systems Analysis Phase", which is enclosed. As we explained, this draft is not complete and we have interrupted work on the report until completion of our structural check.

Very truly yours,

William R. Cox

William R. Cox

WRC/bc
Enclosure

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AD-A163 525

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One of the three reports resulting from the systems analysis phase was a Structural Concept Analyses Report by the A&E. This report was to compare the merits of candidate structural concepts on the basis several factors, including technical & economic feasibility, sensitivity to environmental (Con't)

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design criteria, estimated completion schedule, and risks involved in transportation and installation. Three- and four-pile structures were conceptually designed on the basis of the design data and criteria used for a baseline skirt pile design developed under a previous contract. Also included in the report was previously developed design information regarding a free standing and a modified caisson concept.

ACMR EAST COAST OCEAN STRUCTURES

REVIEW OF SYSTEMS ANALYSIS PHASE

Report To

CHESAPEAKE DIVISION

NAVAL FACILITIES ENGINEERING COMMAND

WASHINGTON, D. C.

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ACMR EAST COAST OCEAN STRUCTURES
REVIEW OF SYSTEMS ANALYSIS PHASE

Report No. 76-10-1

to

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D. C.

by

TERA, Inc.

Engineering Design and Development

Austin, Texas

August 1976

Chesapeake Division (FPO-1)
Naval Facilities Engineering Command
Building 200, Washington Navy Yard
Washington, D. C. 20374

Ref: Contract No. N62477-75-C-0112, Mod. P0002, Engineering Services for Engineering Design Evaluation Support for ACMR Ocean Structures Program

ACMR East Coast Ocean Structures
Review of Systems Analysis Phase

Gentlemen:

Submitted here is our report covering the work accomplished under the above contract modification for technical support to the government in the Systems Analysis Phase (Phase A of the A&E Contract) for the ACMR East Coast Ocean Structures. This report constitutes all the deliverables consisting of the confirming letter reports and the DEA reports. It covers the work described as Work Package 2, Task 1: Environmental Design Criteria, Task 2: Structural Concept Analyses, and Task 4: System Analysis Review.

We have enjoyed participating in such an interesting project and hope we can be of further assistance in the future.

Very truly yours,

W. R. Vines

E. A. Verner, PhD., P.E.
Project Engineer

WRV-EAV/bc
Copies Submitted: ( )

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STRUCTURAL CONCEPT ANALYSIS

Introduction

One of the three reports resulting from the systems analysis phase was a Structural Concept Analyses Report by the A&E. This report was to compare the merits of candidate structural concepts on the basis of several factors, including technical and economic feasibility, sensitivity to environmental design criteria, estimated completion schedule, and risks involved in transportation and installation. Three- and four-pile structures were conceptually designed on the basis of the design data and criteria used for a baseline skirt pile design developed under a previous contract. Also included in the report was previously developed design information regarding a freestanding and a modified caisson concept.

Separate structural concept analyses were performed by the DQA in preparation for a review and evaluation of the A&E's conceptual studies. The original intent of the DQA's structural concept analysis was to facilitate evaluation of the A&E Structural Concept Analyses Report and at the same time to estimate to what extent the structural dimensions and weights will be affected by the choice of pile size. It was proposed that this be done for one structural type only, the four-pile type structure, for the purpose of determining if structural optimization with respect to pile size is an important consideration under the pile drivability constraints imposed by the soil conditions at the ACMR East Coast Sites. Subsequent developments made it apparent that the

impact of possible revisions in the environmental criteria on the Structural Concept Analyses being conducted by the A&E could be so great as to substantially invalidate the conclusion derived from it. These developments were (1) the realization that the actual criteria adopted could result in loads as much as 26% or more higher than the loads being used on The Structural Concept Analyses of the A&E, and (2) the realization that it might not be possible to design a structure of reasonable proportions capable of being installed by controlled driving and jetting under these conditions. ~~As a consequence~~ it was decided to concentrate on the more important task of evaluating the possible impact of higher environmental loads ~~on a structure.~~ in terms of structural size and installation. This was to be accomplished by doing a preliminary design of a four-pile structure comparable to the concept being designed by the A&E and then to increase the environmental forces to the maximum values anticipated and resize the structure to evaluate the impact on structural weight, size and installation problems.

Sensitivity to Wave Force and Pile Drivability

The impact of the wave force criteria and pile drivability assuming a medium size hammer with fairly good availability such as the Vulcan 040 or 060 is indicated by a preliminary sizing attempted.

Assumptions on pile drivability were made based on values given in a supplement to the foundation investigation report.[1] Maximum design penetrations for 30-in., 72-in. and 96-in. O.D. piling were tabulated for each of the borings. Heavy walls (e.g. 1.5 in.) for driving considerations were assumed,

and a value of 0.1 in. for quake at the point was chosen in obtaining the values used for design penetrations. These values were plotted for Site 1 (81 ft MLW) as a function of pile diameter in order to estimate the maximum design penetration as a function of pile size (Fig. 1). Design axial capacity curves for tension and compression were then developed for several pile sizes, and plotted as a function of pile size (Fig. 2). The estimated axial capacities used in the curves were calculated from unit skin friction and end bearing values given in the foundation report.[4]

For preliminary sizing purposes, an attempt was made to estimate the overturning moment as a function of structure width and pile O.D. based on the overturning moment for a similar but smaller four-pile structure previously investigated.[2] In estimating the overturning moment for a larger structure, 50 percent was assumed not to change because the superstructure did not need to be larger, 25 percent was assumed to increase linearly with pile O.D. and 25 percent was assumed to increase linearly with structure width. For each width considered, the required axial capacity was estimated for several O.D.'s and compared to the maximum design axial capacity ~~as~~ shown in the curves ~~developed~~ for ~~axial-tension~~. The results of this rough sizing procedure showed that a structure about 90 ft wide at its base with 72-in. O.D. piles might work. Although the procedure involved necessarily crude approximations of overturning moments for the larger structures, the results indicated that for the loads assumed in this analysis, controlled driving and jetting may not give adequate piling penetrations for a structure of reasonable size.

Comparable Preliminary Design

The conclusions drawn from the rough sizing attempt pointed out the impact of the substantially higher loads used by the DQA as compared to those used by the A&E in their conceptual designs. Five factors, all additive, contribute to the higher loads:

1) Differences in current coupling methods,
2) Differences in Stokes 5th solutions,
3) Different water depths (81 ft vs. 84 ft),
4) Differences in areas for waveload members such as ladders and boat dock,
5) Differences in the treatment of members not perpendicular to the wave.

The amount of net increase resulting from all these factors was later quantified to be an increase of 26 percent. ~~(see Appendix A, pg)~~.

In order to arrive at final results that would be comparable to the A&E's, it was decided to compare wave forces with the A&E by modeling a structure of similar forces and proportions to that being modeled by the A&E. It was understood that the A&E was arriving at a four-pile concept that would be 30 ft wide at the top with 36-in. piles and 36-in. superstructure legs, and it was assumed that it would be otherwise similar to their previous four-pile concept. 2
Preliminary sizes for jacket legs, diagonals, and horizontals, were assumed and a model for wave load take-off was generated. Wave pressures for the 84 ft MLW and the A&E's method of current coupling were generated, but it was, of course, impossible to account for the differences in the Stokes 5th solutions since the cause was unknown.

The total shears and overturning moments from this analysis were compared to those furnished by the A&E for its model. Although neither the

differences in Stokes 5th solutions nor the difference in the treatment of members not perpendicular to the wave could be accounted for in this analysis, the total overturning moment and shear values compared to within 6 percent.

It was then concluded that the following criteria would give similar environmental loads to those being used by the A&E:

1. 84 ft MLW
2. 14 ft total tides
3. 62 ft wave height
4. 12 sec wave period
5. A&E's method of current coupling called "constant volumetric flow"
6. 5.4 fps uniform current at still water level (84 ft + 14 ft = 98 ft)
7. C_d = 0.6 drag coefficient
8. C_m = 1.5 inertia coefficient
9. wind velocity = 150 mph @ 30 ft above MLW, and increased by $(h/30)^{1/7}$ for heights h above 30 ft

Based on these criteria, a structural concept utilizing 36-in. piles, 30 ft apart at the pile top, with 30-in. superstructure legs battered to obtain a 20 ft by 20-ft deck was preliminarily sized.

The analysis was based on a three dimensional model of all the primary load carrying members. Other members, such as stairs, boat landings and annodes were not included in the structural analysis, but all dead loads, wind loads and wave loads for members not modeled structurally were included as concentrated loads. The interaction of the pile inside the jacket legs was treated by assuming the pile was attached to the leg by springs at each of the

shim points, and was completely free of the leg between shim points. For the purpose of this analysis, the additional size of "cans" at the joints were neglected, and members were assumed to extend to the intersection of their centerlines. However, weights were estimated based on face-to-face lengths of members.

The interaction of the structure and the nonlinear pile-soil behavior was approximated in the analysis by assuming a linear pile restraint. The amount of restraint was first chosen to be similar to the restraint observed in similar structures, then modified if necessary for compatibility and equilibrium. The pile-soil interaction was treated separately, using a numerical discrete element model that included the effects of axial load on lateral behavior and considered nonlinear soil restraint. Soil lateral restraint (p-y) curves were developed for the 36-in. piles based on soil properties supplied by McClelland Engineers, Inc.[4]

The analysis of the structure was accomplished using STRESS. Individual members were checked for strength using the AISC design code as modified and recommended by API RP 2A (1975). For the purpose of this conceptual design, the members were checked at their ends only. Modifications to member wall thicknesses and/or O.D. were made on the basis of these checks. A second waveload analysis showed negligible change in total shears and overturning moments due to changes in member sizes.

Pile lengths were chosen from axial capacity plots to resist the maximum axial loads, and trial wall thicknesses for the pile below the mudline were

selected for the maximum shear. In the design for lateral loads, the generated p-y data was used to simulate the response of the soil in a nonlinear discrete element modeling of the soil-pile system. A stress wave analysis of pile drivability was made which indicated that the piles with minimum wall thickness for stress did not have acceptable drivability even with a Vulcan 060 hammer assuming a quake at the point of 2% of the wall thickness. Drivability of a heavy wall pile (36 in. x 2.000 in.) was indicated to be fairly good d~~rivabilit~~y. A later analysis using a quake at the point of 0.1 in. indicated marginal drivability of the heavy wall section.

Revised member sizes are shown in Fig. 3, and required piling length and wall schedule are shown in Fig. 4.

A comparison of Fig. 3 with the structural concept shown on page 12.07 of the A&E report[7] indicates fairly close agreement in member sizes except for the superstructures which are substantially different in width and for the horizontal bracing which generally is controlled by the designer's choice of an allowable slenderness ratio rather than load carrying capacity. The jacket weight for the structure shown in Fig. 3 is about 276 kips which is 13% lower than the jacket weight reported by the A&E, mainly due to difference in horizontal bracing and possibly can sizes and corrosion allowances. The superstructure weight of 144 kips is considerably lower than the A&E's 172 kips (16%) due mainly to the different deck width. The A&E does not show a piling weight required for stresses only but it is noted that the piling shown is about 16% lighter than the 36 in. x 2 in. piling mentioned above which showed fairly good marginal drivability.

Effect of Higher Environmental Loads

Surprising.
Information receiv . from the A&E regarding the results of their sensitivity study of environmental factors indicated a large difference in

the forces given by the two wave and current coupling techniques along with a lesser but possibly additive difference in force given by the two wave theories. It was estimated that there was an increase of about 23 percent between the base shears and moments obtained using Glenn's wave theory with the "constant volumetric flow" current coupling technique and those obtained using Stokes 5th wave theory with a "riding wave" current coupling technique in 105 ft MLW (See Environmental Design criteria section). This large difference in forces raised the question of the effect of the increased forces on structural weight and installation. Therefore, it was decided to study the possible differences in forces and weights due to the change from the old criteria used as the basis of conceptual design to the most severe criteria which might be adopted for design. The changes would apply to several factors including: the water depth, from 84 ft to 81 ft MLW; the wave criteria, from the 62 ft, 12 sec wave used in the original conceptual studies to the 50-year wave and current criteria recommended by A. H. Glenn; wave theory, from Stokes 5th to Glenn's theory; and current coupling technique, from the "constant volumetric flow" to the "riding wave" current coupling method.

Velocities for one of the waveload cases, Glenn's wave theory with the "riding wave" method of current coupling, were not available for the 81 ft MLW depth. However, this information was available for the 105 ft MLW depth. A comparison was made at that depth between the velocities given by the desired combination and those given by direct superposition of the current and wave particle velocities, which showed that the velocities for Glenn's wave theory with the "riding current" coupling technique were an average of 6 percent higher than those from direct superposition of the current and wave

velocities. Wave pressures in 81 ft MLW were then obtained by increasing drag pressures obtained by direct superposition by 6%.

To confirm that Glenn's wave theory and drag coefficients with the "riding wave" method of current coupling would give the maximum environmental loads, for the 81 ft MLW site as it did for the 105 ft MLW site, wave load base shears and moments were calculated for each wave theory and current coupling method being considered. As shown in Table 1, the maximum forces are predicted with the approximation of Glenn's wave theory with the "riding wave" coupling technique.

In order to predict the magnitude of the effect of the increased forces on structural weight, the structure in 81 ft MLW was analyzed using this maximum waveload case. Revisions were made to the member sizes, and the pile schedule, and are shown in Figs. 5 and 6. The net increase in wave forces from that based on the conceptual design criteria was 23%, with a 25% increase in moment. Increases in the weight of structural steel amount to about 26% with most of it in the piling (41%) as shown in Table 2. The piling weights shown in Table 2 do not reflect the additional steel that might be required for installation in the form of extra wall thickness for driving stiffness or insert piling.

Drivability. During the pile driving analysis information was received that indicated the possibility that nonconservative assumptions had been made for the elastic ground compression (quake) at the tip of the pile. Initially, the value for quake at the tip for an unplugged condition was assumed to be 0.02 times the wall thickness at the pile bottom, as suggested by McClellands' drivability report[1]. Dr. Lee Lowery, of Texas A&M University, a recognized consultant on pile drivability, expressed the opinion that the

original value of 0.1 in. used by Smith will give better results than assuming the quake proportional to wall thickness or diameter.

Additional stress wave equation analyses using 0.1 in. for quake at the pile tip resulted in substantial reductions in the predicted drivability of the 36-in. O.D. piles. The additional analyses predicted that the wall schedule needed for stress would not drive, and that the margin of drivability for a wall as heavy as two inches was slim even with the use of a Vulcan 060 hammer.

Possible solutions to the drivability problem would include, among others, (1) increasing structure width, (2) using a bigger hammer, (3) using a heavier pile (eg. 42 in.) to set added drivability due to increased stiffness, and (4) using insert piles.

Review of A&E Structural Concept Analysis Report

The Structural Concept Analysis Report[6-9] prepared by the A&E presents fairly detailed conceptual designs for standard templet type structures of three piles, four piles, and four small piles with four skirt piles. An attempt at preliminary optimization was made in initial sizing of the three pile and four pile concepts developed for the report. The previous skirt pile design which was a "quick fix" for a previous four pile design was used as the basis of comparison; the original four pile design being judged uninstallable after soil borings were obtained. Other "quick fix" preliminary caisson designs by the A&E were presented in the form in which they stood prior to beginning the structural concept investigation.

The conceptual designs for the templet type structures were based on the primary design driver which is strength and maximum storm load. Consideration of earthquake and fatigue loading were properly omitted from the conceptual design considerations. Proper consideration was given to pile-soil interaction, pile drivability, pile-structure interaction, and joint strength design, as well as member strength design.

In determining pile size and structural width, the A&E conducted a weight optimization study for the three pile structure. This considered piles of 36-in. diameter to 42-in. diameter and widths from 50 to 90 ft on a side at the base. Over this range of parameters the structural weight (consisting of more than 50 percent piling) varied less than 18 percent. Basic

assumptions were:

1. Wave forces did not vary with pile size or base size.
2. Weight of bracing was calculated from total length and estimated average brace sizes increasing with base size.
3. Jacket leg wall thickness and cam sizes did not vary with pile size or base size.
4. Piling wall thickness did not vary with pile diameter.

Of these assumptions, the estimation of jacket leg size (No. 3) is probably the most realistic, and the calculation of bracing weight (No. 2), of course, depends entirely on the method used to estimate the average brace size, which was not explained. The assumption of constant wave force (No. 1) in determining pile size and penetration is probably not too bad since the piling size would be the most affected by changes in wave force, except for the fact that driving considerations controlled the piling wall thickness in this case. The assumption of constant piling wall thickness with pile diameter is probably the least tenable assumption considering that the piling accounts for over 50 percent of the weight, and the total range of weights was only about 18 percent and even less for a given base size. Since the pile wall thicknesses are largely controlled by drivability, and it is fairly well established that pile drivability depends on pile cross-sectional area rather than wall thickness, it would seem that increasing pile diameter should not increase piling weight per unit length in this case, and only slightly increase overall piling weight due to larger wave forces and the resulting increased penetration. In summary,

it would appear that the variation in weight with respect to pile diameter is probably not as great as indicated by the study, and that, due to the relatively small (10%) weight variation with a 60 percent variation in base size, the optimization with respect to base size is probably not very critical compared to the increases in transportation cost for larger base sizes and increases in pile driving problems for smaller base sizes.

Optimization with respect to base size for the four pile concept was based on similar assumption but for the 36-in. pile size only. This case showed even less variation with base size and is probably even less critical compared to the transportation and drivability considerations.

The Structural Concept Analysis Report[6-9] prepared by the A&E in general accomplishes the objectives of comparing the candidate structural types (3-pile, 4-pile, 4-pile with skirt piles, free standing caisson, and braced caisson) with respect to feasibility, cost, sensitivity to environmental criteria, installation risk, and schedule.

It was noted in the report that the only question of technical feasibility for any of the candidate concepts is the ability to drive the piling. The risk factor or probability of failure to install the structures was evaluated subjectively rather than numerically. At present there has apparently been no work published that would establish a probability density function for wave equation predictions. All structures were designed for the same loading criteria and resistance criteria.

This, of course, does not mean that the

reliability of each structure would be identical, because the probability of failure distributions for different components (e.g. joints, members, piles, soil, etc) are not uniform, giving different net failure probabilities for different structural concepts designed under the same resistance or strength criteria. However, this does not affect the cost comparison significantly, since the caissons are not evaluated on the basis of cost, and the templ~~ate~~et type structures are similar enough to assume they would all be of similar reliability.

Sensitivity to environmental design criteria was evaluated on the basis of displacement at the lower deck under the design load. It was noted that the caisson concepts showed deflections of well over a foot under design loading whereas the multiple concepts showed less than a foot deflection. It is obvious that the caisson concepts are much more sensitive in terms of deflection than the other concepts. However, the question of accepability under deflection criteria for the operating wave condition was not addressed, nor was the question of sensibility of designing caissons to satisfy deflection criteria.

The evaluation of transportation and installation risks was also made on a subjective basis. It was noted that the templet type structures "do not present any problems which are not ordinarily encountered with this type structure" which, of course, applies equally well to the caisson structures

and says nothing. These structures are also subject to overturning during the early phases of installation, as are the caissons. In fact, due to the larger number of piles to be driven the resistance to overturning might not attain its full value in a short a time as the caisson structures. If the caisson structure could infact be driven to grade in one continuous operation without the necessity of an add-on section, the installation time might be fairly short compared to the duration of reliable weather prediction. On the other hand, the jackets without piling might be capable of withstanding sufficient wave force to preclude any concern about the length of time required to install the piling. Without some attempt to establish qualitatively the risk involved it is not apparent that either structural type would be subject to sufficient risk of overturning during installation to make it a significant basis for comparison. Without estimates of installation time, weather predictability, and other rational analysis it is difficult to support the opinion that the caisson structures represent an unusual risk or otherwise.

As was noted in the report, schedules for the three templet type structures are comparable and do not represent a significant basis for comparison. No reason comes to mind that the same conclusion would not hold for the caisson structures as well.

In summary, the recommendation of the three pile concept as the best of the three templet type structural concept candidates is based on the results

of a comprehensive design effort showing a cost saving of 31 percent over that of the skirt pile concept. Although the three templet concepts are not necessarily of equal reliability, the redundancy of the system and the unlikelihood of complete failure would probably result in an insignificant cost of risk differential. Caisson concepts were ruled out as having excessive deflections and installation risks on the basis of preliminary calculations and judgments of the A&E.

DRAFT

ENVIRONMENTAL DESIGN CRITERIA

Introduction

One of the primary purposes of the Systems Analysis Review Phase of the A&E contract was to establish the environmental design criteria and methods of predicting the resulting forces on the structures. The required environmental design criteria consists of (1) the wind, wave, tide, and current characteristics associated with storm recurrence intervals of 25 and 50 years, (2) a 20 year wave spectrum for fatigue analysis, and (3) normal monthly wave spectrum for one year. The method of predicting force on the structure can be thought of as consisting of three parts, (1) a wave theory for predicting water particle velocities and accelerations, (2) a theory for coupling the wave velocities with the current velocities, and (3) a theory for predicting wave force on a cylinder as a function of water particle velocity and acceleration. For each of these three parts, there are various and substantially different methods in common use in the design of offshore structures.

Review of A&E Environmental Design Criteria Report

Recommendations were made by the A&E with respect to all of the above requirements in the Environmental Design Criteria Report.[10] The recommended environmental design criteria comprising the basic wind, wave, and current predictions are derived from Appendix A of that report by A. H. Glenn, a recognized oceanographer. These predictions are not under review by the DQA and are accepted as the criteria to be used in review of certain specified methods of calculating forces on structures.

(INCOMPLETE)

REFERENCES

1. Supplemental Report

2. Structural Check

3. Soil Report, Vol. 1

4. Soil Report, Vol. 2

5. Structural Concept Report (Crest)

6. Appendix A

7. Appendix B (4-pile)

8. Appendix C

9. Appendix D

10. Environmental Design Criteria Report

TABLES

TABLE 1

WAVE LOADS FOR 81 ft MLW

TABLE 2

EFFECT OF HIGHER ENVIRONMENTAL LOADS
ON STRUCTURAL WEIGHT

Item	Structural Weight (Kips) Conceptual Design Criteria	Maximum Anticipated Criteria	% Increase
Jacket	276 kips	315 kips	13
Superstructure	144 kips	159 kips	10
Piling* Req'd for stress	366 kips	515 kips	41
TOTAL	786 kips	989 kips	26

*Probably not drivable with Vulcan 060 hammer without substantial additional steel for driving stiffness.

FIGURES

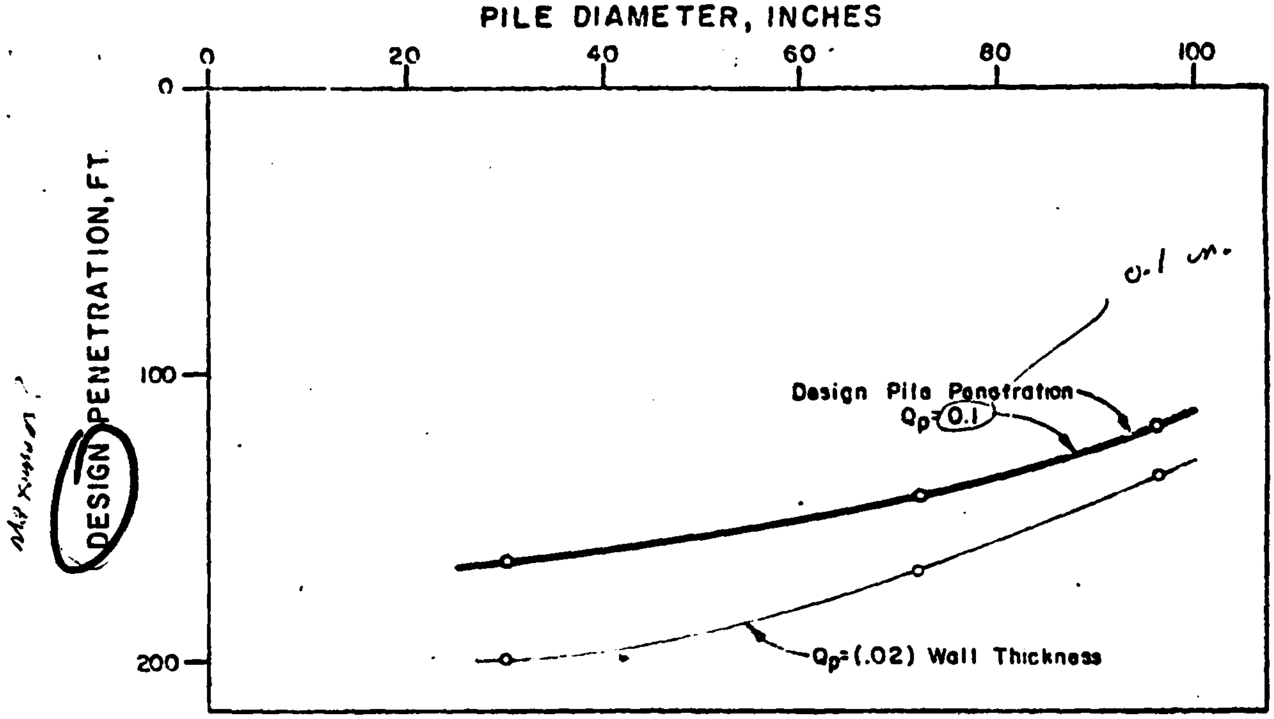


FIG. I. MAXIMUM PILE PENETRATION ESTIMATES VULCAN 040 AND 060 HAMMERS

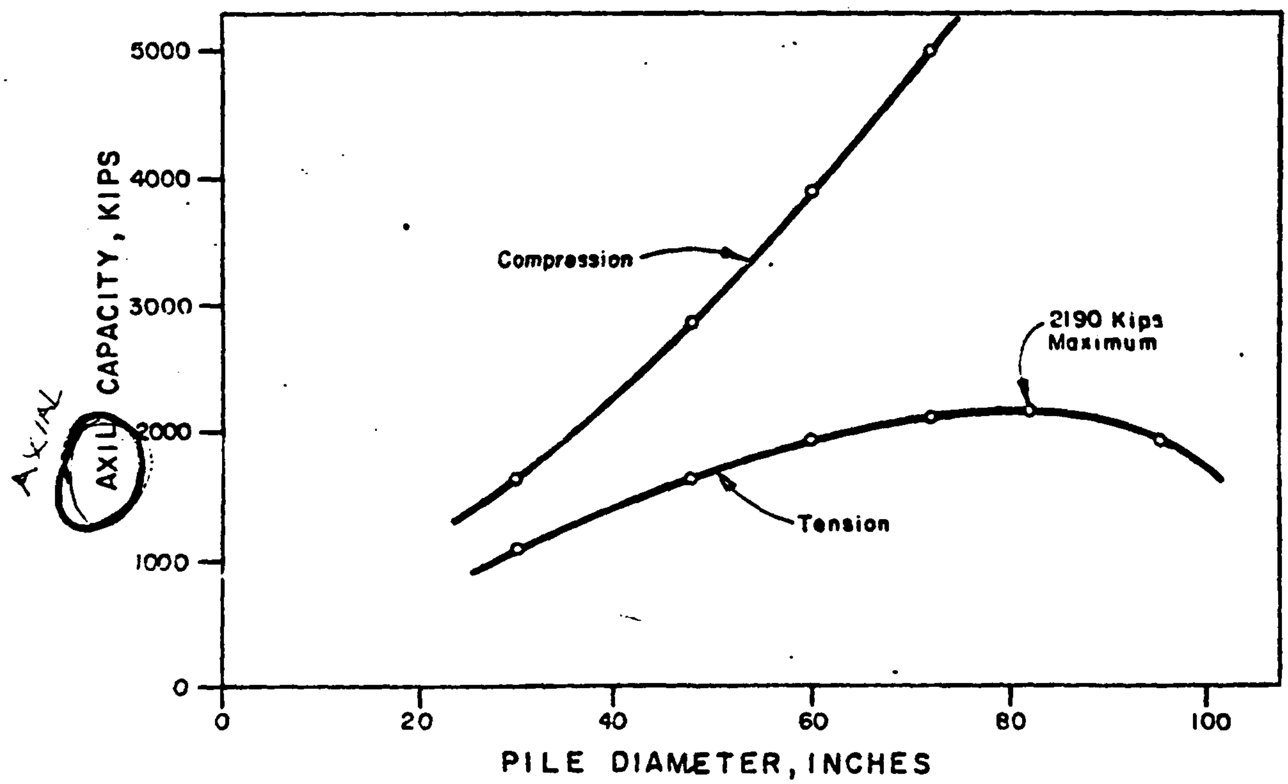


FIG. 2. PILE CAPACITIES FOR DESIGN PILE PENETRATIONS

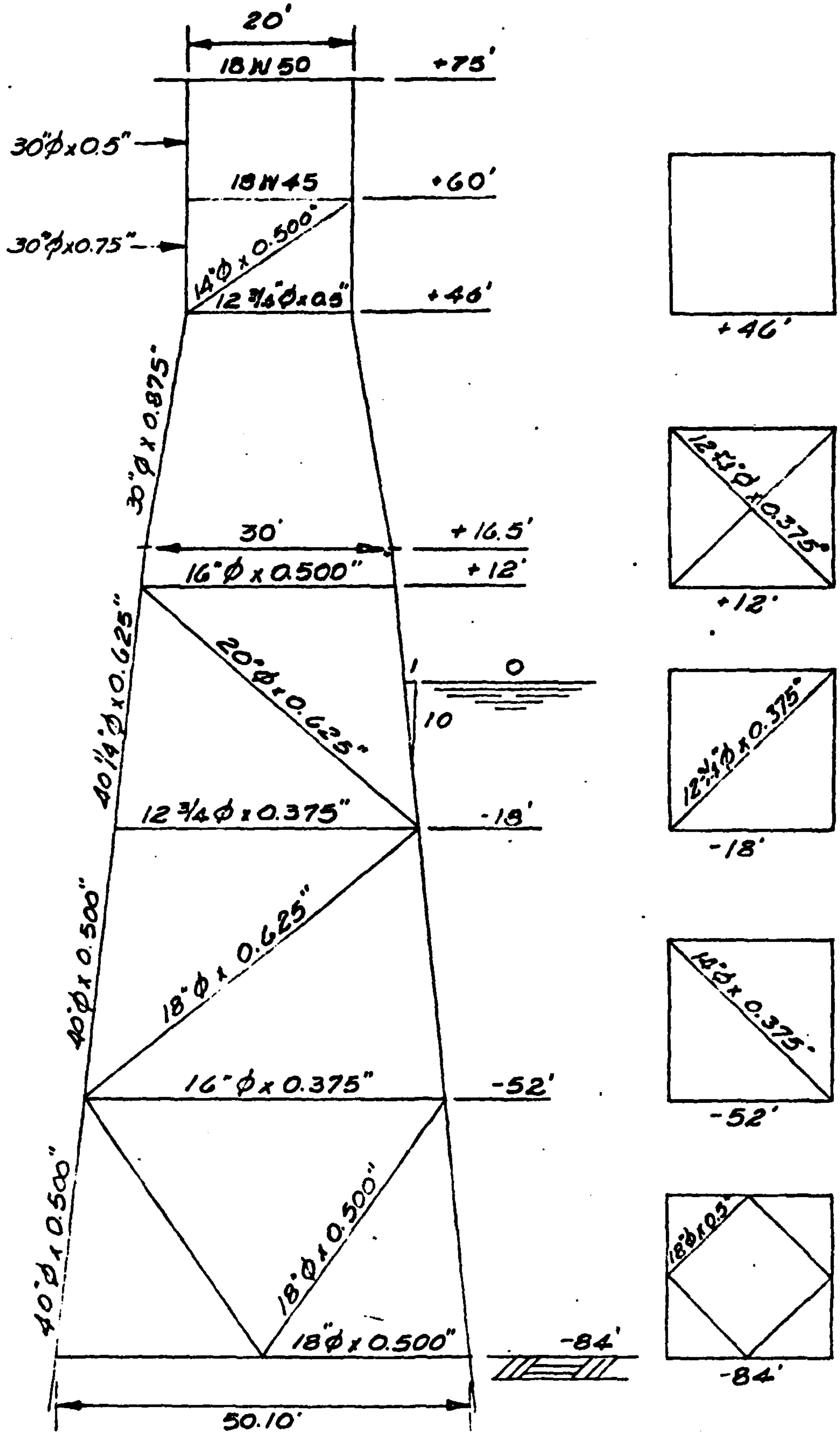


FIG. 3. FOUR PILE CONCEPT, CONCEPTUAL DESIGN CRITERIA

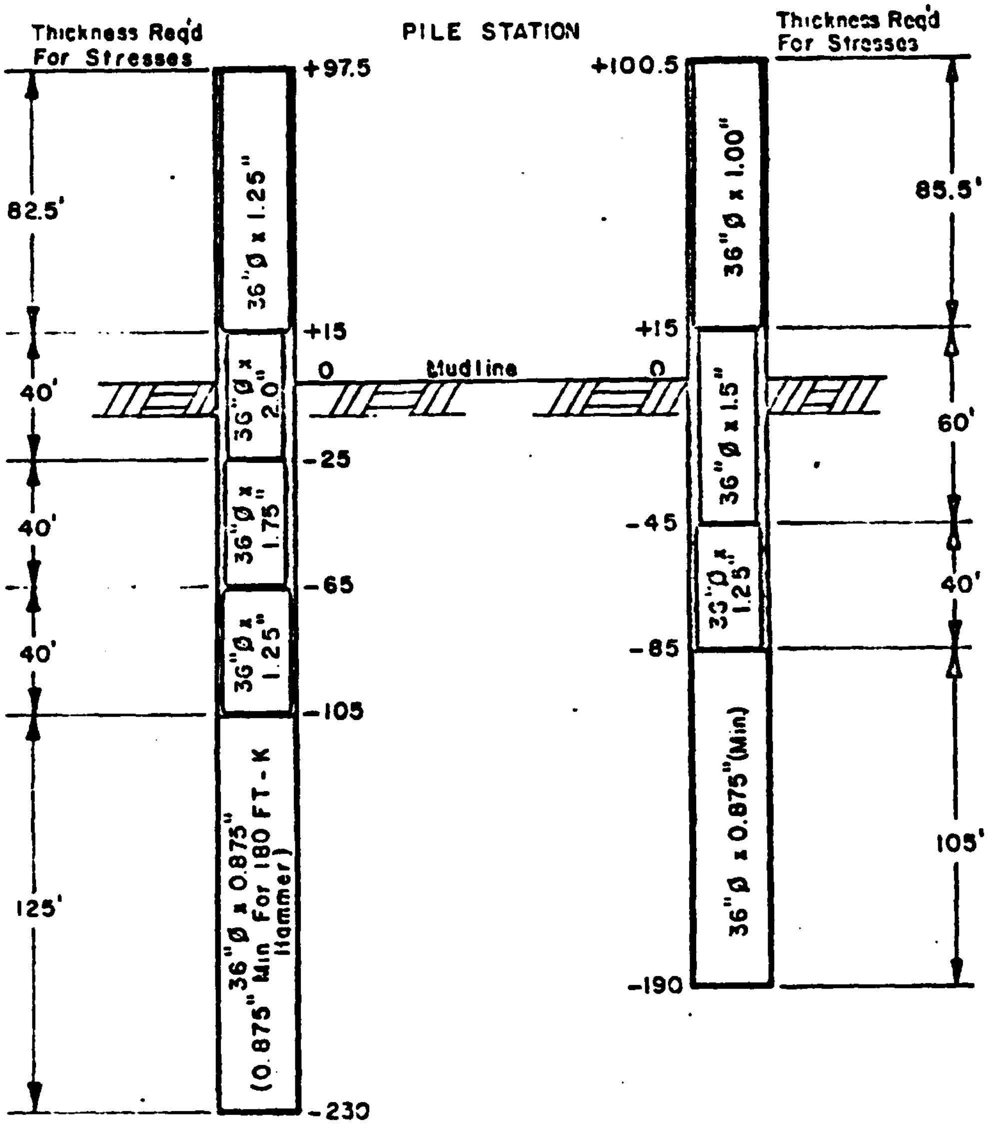


FIG. 5.
PILE SCHEDULE FOR STRENGTH, MAXIMUM ANTICIPATED CRITERIA

FIG. 4.
PILE SCHEDULE FOR STRENGTH, CONCEPTUAL DESIGN CRITERIA

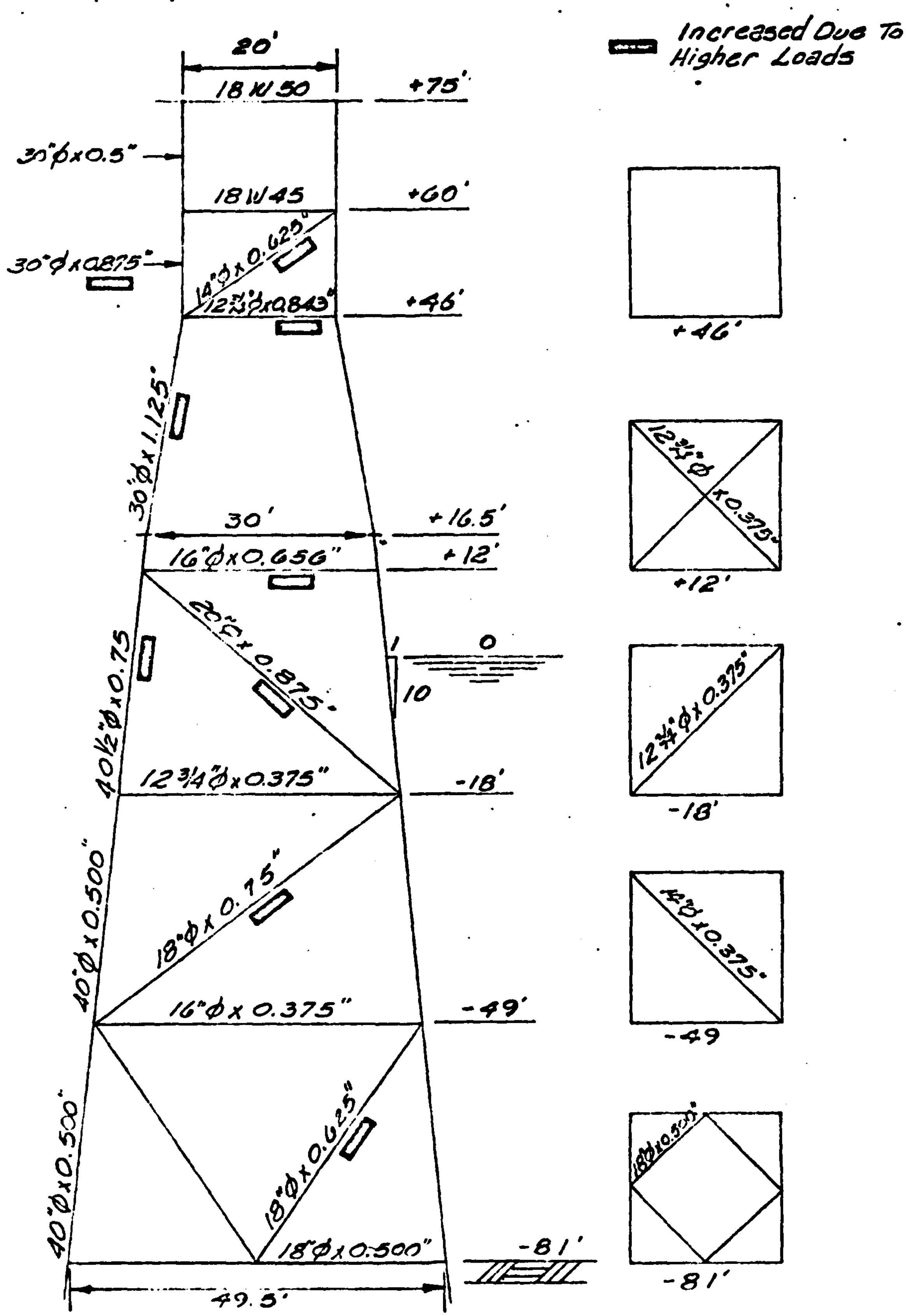


FIG. 6. FOUR PILE CONCEPT, MAXIMUM ANTICIPATED CRITERIA

APPENDIX A

NOTES ON SYSTEM ANALYSIS REVIEW MEETING

Form 111-6

OFFICE MEMORANDUM

File: T-76-10 Date: July 8, 1976

From: Ed Verner Time:

To: File Via

NOTES ON SYSTEM ANALYSIS REVIEW MEETING

Monday, 24 May 1976

Present:

Navy	Crest	TERA
Bodey	McCann	Cox
Erchul		Verner
Kim		Vines
Ling		Dean (Consultant)
Raecke		
Brill		
Masso		
Scully		

After Commander Erchul's opening remarks, Bodey introduced meeting noting the purpose to review, discuss, and resolve all issues involved in and resulting from the Phase A: Systems Analysis Review conducted by Crest Offshore. Bodey provided some notes (Attachment A) which summarized the objective, scope, five specific questions to be answered, a cartoon indicating the background requiring the System Analysis Review, and an agenda for the meeting.

John McCann summarized the results given in Crest's Environmental report. It includes:

1. Storm parameters for water depths of 81 ft, 93 ft, and 105 ft with recurrence intervals of 25 and 50 years (by A. H. Glenn).
2. A 20 year wave distribution for fatigue analysis (by A. H. Glenn).
3. Monthly wave distributions for analysis of installation risks (by A. H. Glenn).

4. A load sensitivity study for the four pile structure with skirts previously designed. The result was to show the forces and moments on the structure as determined by the following combinations of wave theory and current coupling theory:

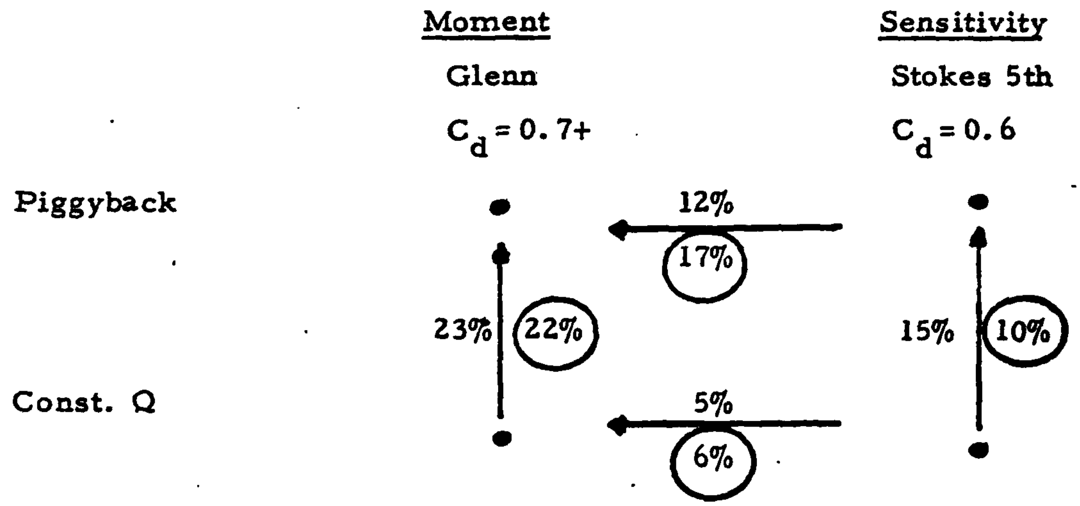
Wave Theory	Current Theory
Glenn	Constant Volumetric Flow (Const. Q)
Stokes 5th	"
Glenn	Riding Wave (Piggyback)
Stokes 5th	"

McCann noted that they had discovered an error in Crest's Stokes 5th solution for the particular conditions under consideration. For the 105 ft MLW site with 14 ft tides, 62 ft wave, 12 sec period but no current, he found that the drag pressure at the crest with their program was 525 psf while TERA had 577 psf and Synercom had 581 psf. He said Crest had previously checked their program only for deeper water for which it functions properly.

Dean expressed concern that the Const. Q method is not applicable, especially for the case of constant current. Kim said there must be some adjustment at the boundary. Kim noted that the method of applying Morrison's equation to indirect members can make differences on the order of 20% in waveloads(OTC Paper No. 2723 by Wade and Dwyer, 1976).

Verner presented a summary of a load sensitivity study (out of scope) done by TERA for the 81 ft MLW water depth for comparison with that done by Crest for the 105 ft MLW water depth. The sensitivity of the overturning moment is most significant since it directly affects the critical pile penetration problem. The numbers were presented as shown below. It ~~was~~ noted that the Glenn-Piggyback pressures were obtained using direct superposition and increasing the resulting drag pressures by 6%, which is the average increase observed for piggyback over direct superposition

for Glenn's wave in 105 ft MLW. Circled numbers are those predicted by Crest for the 105 ft skirt pile structure, others are those presented by TERA for a four-pile structure in 81 ft MLW.



Dave Raecke presented a sensitivity study of a single 12 in. pile in the 105 ft water depth for Glenn's wave:

	Moment (ft-lb)	Force (kips)
w/o current	3.3×10^6	31.4
w/current: Const. Q	4.2×10^6	39.9
w/current: Piggyback	5.2×10^6	49.9

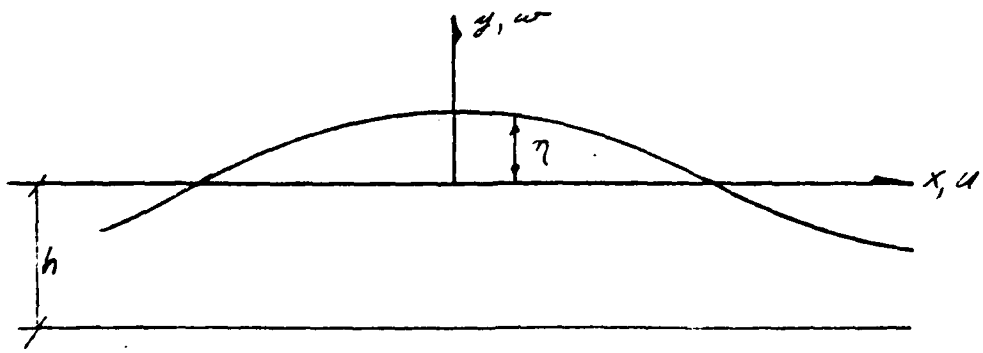
which shows a 24% increase in moment for the piggyback current coupling technique which agrees with 22% and 23% values obtained for complete structures above.

Mr. Bodey asked TERA to explain what they had done in the way of evaluating the wave theories and current coupling techniques. Verner explained that the Piggyback current coupling technique resulted from the previous current profile which was constant with depth. In checking the previous designs of Crest Offshore, TERA had maintained that the piggyback method is correct for the case of constant current with depth. The present criteria supplied by A. H. Glenn specify a nearly linear current and for that case TERA recognizes that neither the Piggyback method or the Const. Q is correct. Although Crest Offshore was evaluating the effect on

the wave force of using the different current coupling techniques, TERA felt that this would only indicate the significance of the problem. Should the result be significant, it was felt that some more rational method or experimental data would be required to determine which method was the more appropriate.

In addition to the current coupling question, there was also a question of whether Glenn's nonlinear wave theory or Stokes 5th wave theory would be more appropriate. This was a question raised by the Navy and also one that TERA did not feel competent to comment on. Hence, TERA had obtained the services of a recognized oceanographer, R. G. Dean, who has published considerable work on the applicability of wave theories to different situations and has been involved in research on a wave theory which is based on a numerical solution in terms of the stream function formulation, and includes a linear current in the basic equations which are formulated on the basis of conservation of vorticity. TERA had asked Dean to evaluate the four different wave theory/current coupling methods on the basis of satisfaction of the free surface boundary conditions (FSBC) and the pressure profiles, and to compare them with his numerical stream function solution.

Professor Dean presented his results in the form of a table (Attachment B). He apologized for its rough form noting that he had only just completed the calculations that morning. The table shows moments and forces on a three foot diameter pile due to both drag and inertia forces using $C_d = 0.74$ and $C_m = 1.34$ for all theories. Inertia forces and moments are for a phase angle giving about the maximum, and even then are very small compared to the drag forces with the crest of the wave at the pile. Also shown is the range of the FSBC's which were described by Dean:



At the free surface $(y = \eta)$

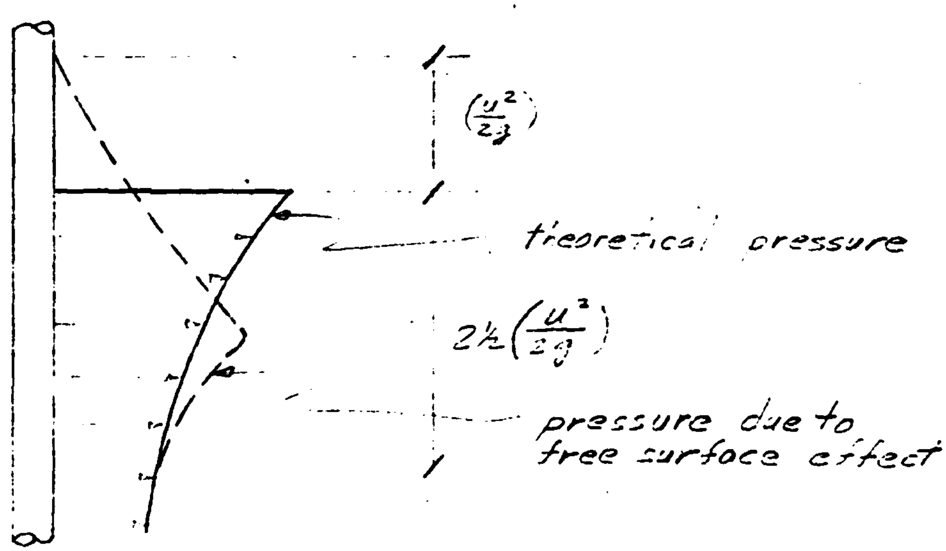
1. $$\frac{\partial \eta}{\partial x} = \frac{w}{(u + U_c - C)}$$ (Kinematic)

2. $$\eta + \frac{1}{2g}\left[(u+U_c-C)^2 + w^2\right] = const.$$ (Dynamic)

Professor Dean noted that except for the numerical stream function solution, the ranges of the dynamic FSBC were so large that he felt that the difference between one case and another case would not necessarily be indicative of which case gave a better solution. Since the stream function solution gave a good fit to the boundary condition, he felt that a comparison of pressures and velocities at the crest with the stream function as a standard would be most appropriate.

Dean noted that he has analyzed a great deal of hurricane data (wave force projects I and II) with waves as high as 39.4 ft, but none as high as the 60+ ft waves under consideration. He feels that all this data only gives support for the numerical stream function solution water particle velocities. The tabulated forces and moments for a three foot diameter pile represent both pressures and velocities since the same drag coefficient was used for all cases.

He explained with the following diagram the "free surface effect" which allows better correlation of measured force with theory:



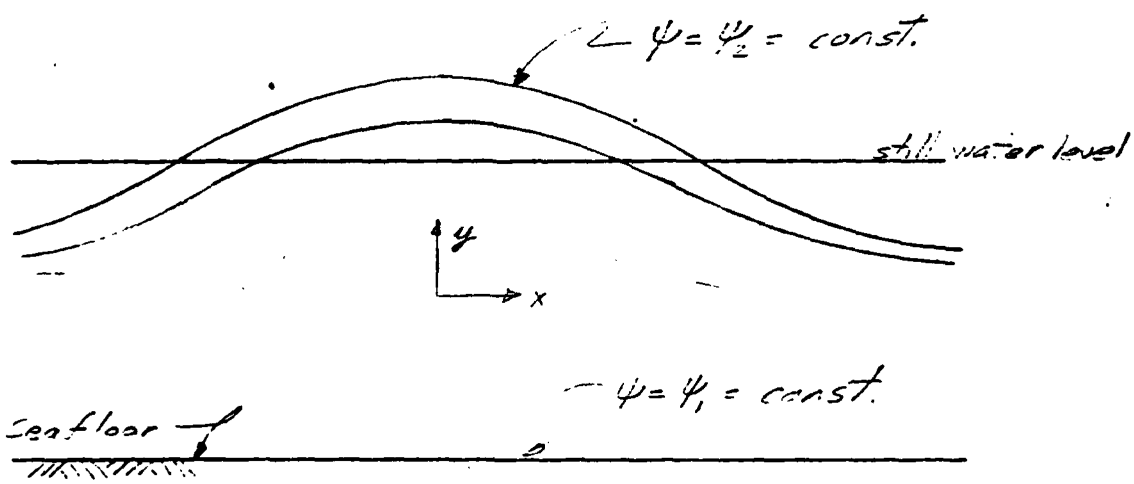
He noted that this would probably amount to a reduction of about 20% in moment and 10% in force for a monopile in conditions similar to those under consideration.

As far as drag coefficients are concerned, Dean noted that Glenn used about 0.66 for a 12 in. pile with an increase for a 36 in. pile which would indicate about $C_d = 0.74$. Dean felt that this is about what he would have predicted and hence all his comparisons were made on that basis.

Dean was asked about the method of dealing with current in the stream function solution. He noted that for no current or constant current, the equation of motion is

$$\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} = \nabla^2 \psi = 0$$

where ψ is the stream function whose partial derivitives with respect to x and y give the particle velocities. The free surface and the seafloor are both lines of constant stream function value (stream lines) as shown below.



For the case of a nonuniform current, the basic assumption is that the vorticity must be constant along a streamline. The differential equation becomes

$$\nabla^2 \psi \quad = \text{vorticity} = f(\psi).$$

For the case of constant current, the vorticity $f = \frac{\partial U_c}{\partial y}$ is zero resulting in the LaPlace equation. For the case of a linear current, the vorticity is constant resulting in the Poisson equation.

Dean noted that the petroleum industry is presently obtaining experimental data which may be able to experimentally validate this theory, but it is not experimentally validated at present. He noted that since the total velocity

$$U_T = U + U_c - C = -\frac{\partial \psi}{\partial y}$$

resulting from the above formulation includes the current velocity U_c implicity, then

$$Q = \int_1^z U_T \, dy = \int_1^z -\frac{\partial \psi}{\partial y} \, dy = (\psi_1 - \psi_2) = \text{const.}$$

the flow across any section is constant as shown above since ψ_1 and ψ_2 are constant. Thus, the stream function solution in addition to conserving vorticity, gives the continuity of flow which both the Piggyback method and the Const. Q methods attempt to maintain.

Cox asked if Dean felt that the velocity and acceleration profiles are realistic for the ACMR site. Dean said he had not looked at Glenn's wave statistics and could not comment on their validity. He predicted that when Shell releases some data obtained for a hurricane passing at 10 miles distance, we may be surprised at values of measured current.

Having discussed the basis of the comparison presented by Dean, several minor points were discussed. Bodey got confirmation from Dean that no free surface effects were included in his stream function values presented in the table and suggested that such a reduction might bring the prediction more into line with the Glenn wave theory with the Const. Q coupling technique. Dean noted that the effect of higher order wave theories is to peak the wave surface profile and give higher velocities near the surface.

Dean also noted that the Const. Q coupling technique did not give continuity along the free surface since, for the case of a uniform current, the control volume for continuity across the vertical plane is moving with the wave at celerity C_o which would render the profile motionless in still water. With the addition of horizontal current velocities (modified or otherwise) to the wave partial velocities, the surface of the wave would move out ahead of the control volume. This is true to some extent with the piggyback, but in that case, the celerity with which the control volume is moving for continuity along the vertical surface is $C_o + U_c$ (avg) where U_c (avg) is the average current value between the crest and trough, so that although the surface of the wave would be distorting, it would not be moving out grossly ahead of the control volume.

Dean noted that he felt that the stream function solution was empirically in best agreement with the Stokes 5th theory with piggyback coupling based on the pressure profile at the crest assuming a $C_d = 0.74$ was used rather than $C_d = 0.60$ as was previously used.

After breaking for lunch, Glenn's comments on current coupling techniques were discussed. In general it was concluded that his wave height and period predictions and current predictions were independent of any wave theory and current coupling technique and could be used regardless of what theory was used to predict wave kinematics and forces. Glenn's notes indicated that his predictions were not for waves moving into an area of currents and that the current should not affect the wave height or wave period. Verner pointed out that the piggyback method

considers the effect of a wave which exists simultaneously with a current and does not affect the specified period or wave height, but does affect the wave length. McCann noted that Glenn is modifying his coupling technique and thinks the correct coupling method is between Piggyback and Const. Q.

Verner suggested that since it would ultimately be the Navy's and not Crest's decision on what wave pressures to use, and since it was generally felt that the stream function approach indicated by Dean was the better approach, that Dean might be asked to supply wave pressures based on the stream function with some allowance for the reduction due to the free surface effect. Dean was asked to consider whether he might be able to provide that service.

The subject of wave forces on the marine growth was brought up and McCann noted that API says that this is not a problem on the East Coast. It was noted that there may be Navy specifications which would apply to this area. Crest did not allow for marine growth in their calculations. Dean was asked to review again the numerical stream function solution and the basis for including current or vorticity, which he did.

Sometime earlier, the differences between Crest and TERA wave forces had been noted for the original four-pile design by Crest:

	Crest	TERA	Percent Increase
moment	106,665 'k	134,197 'k	26%
shear	1,275 k	1,604 k	26%

Tuesday, 25 May 1976

Present

Navy	Crest	TERA
Bodey	Johnston	Cox
Erchul	McCann	Verner
Masso		Vines
Ling		
Brill		
Raecke		

Bodey began the second day of this meeting by recapping the events of the previous day: the presentations of the work done on environmental questions by Crest and by TERA, comparisons of sensitivity to the wave theory and current coupling used; Dean's discussion of the Stream Function, its possible modifications and comparisons to other theories; and the decision to use the Modified Stream Function if Dean is available.

McCann asked what would happen if Dean was in fact not available. Bodey replied that in that case Glenn's wave theory with constant Q current coupling would be used. No comment was made by TERA. McCann indicated Crest's wish to use a 25 year return period if the Modified Stream Function wave theory were employed. Bodey pointed out that 25 year criteria results in only about 10 percent reduction in forces, and established that Government's position was to defer any question of reducing criteria until later.

McCann presented Crest's Structural Concept Analysis Report. He stated that the purpose of the report was to compare structural weights, costs, technical feasibility, scheduling and transportation and installation risks. Preliminary designs were generated for three-pile and four-pile structures and previous work was reproduced for skirt-pile and two caisson concepts. In the preliminary design of the three- and four-pile structures, optimization of structure size was obtained by a comparison of approximate weights versus pile spacing for several pile sizes. The effect of additional weight of extra wall thickness required for driving was not included in these approximations. After the base dimensions and piling had been selected, the preliminary design was refined using Synercom's Seaload and STRAN

programs. The comparison of the three-pile, four-pile, and four-pile with skirts, indicated that the three-pile was most desirable at about 70 percent of the weight of the skirt pile. The caisson and modified caisson were ruled out on the basis of unusual installation risks, large deflections, and sensitivity to dynamic amplification.

Verner presented a brief summary of the structural concept analysis performed by TERA in parallel with Crest. He explained that the purpose of the structural concept analysis was two-fold: (1) to insure that there was initial agreement between the A&E and the D.Q.A. before initiating the design phase, and (2) to more fully investigate the effect of pile size for one particular structure. It was pointed out that these structures are unusual in that the overall width is determined by structural considerations only, and is highly sensitive to overturning moment because axial pile capacity is limited by driving considerations.

The second objective was replaced by a more important objective when it was realized how much different Crest's conceptual design forces were from those being used by TERA (about 26 percent more moment and force had been calculated by TERA in checking the original four-pile design of Crest's). It became apparent that the real question that might remain at the end of the Systems Analysis Review would be to assess the impact on the actual structure and piling if higher wave pressures were adopted.

Verner noted that the first step in getting on track with Crest was to do a wave load takeoff on a structure similar to that being modeled by Crest using as nearly as possible the same criteria, wave theory and current coupling theory. TERA received from the Navy the information that Crest was modeling a four-pile that was 50 ft wide at the base with 36 in. piles and superstructure legs, using a Stokes 5th wave theory with Const. Q coupling, and the 62 ft, 12 sec wave, but the old water depth of 84 ft instead of 81 ft, since the 84 ft depth had been used for the baseline skirt pile structure. TERA tried to model these conditions as closely as possible but, of course, had only minimum information on the structure and could not model the error in Crest's Stokes 5th solution. Despite these factors,

the overturning moment calculated by TERA was only 6 percent higher than that reported by Crest, thus indicating that the use of the 84 ft MLW and the Const. Q current coupling resulted in wave forces only slightly higher than those being used by Crest.

TERA then was confident that they were on a common basis with Crest and proceeded to check out member sizes for a structure of comparable dimensions but with 30 in. deck legs instead of 36 in. legs. The member stresses were checked at member ends only and the resulting member sizes were indicated (Attachment C). It was noted that member sizes were essentially the same as those given by Crest except that the lightly stressed horizontal members were somewhat lighter, which was not surprising since they are primarily controlled by the slenderness ratio and the designer's philosophy. The only other differences were the 30 in. deck legs, the deck leg batter, and the use of diamond bracing at the bottom level of horizontal framing.

Verner explained that the next objective was to determine the impact on the structure size and weight of the highest likely wave load to be selected as a result of the Systems Analysis Review phase. It was estimated that this would be the Glenn wave theory and drag coefficients with the Piggyback current coupling. It was noted that these wave pressures were not available for the 81 ft MLW site, so Glenn's wave with direct superposition was used with a 6 percent increase in drag pressures which was the average difference in drag pressures at the crest for the Piggyback method and direct superposition in the 105 ft MLW site. For the structural concept shown in Attachment C, this approximation to Glenn's wave with Piggyback coupling gave 25 percent more overturning moment and 23 percent more horizontal force. The resulting member sizes were indicated (Attachment D). As far as the jacket and superstructure were concerned, it was noted that only wall thickness changes were required. Percentage increases in steel amounted to 13 percent in the jacket, about 23 percent in the main superstructure members and 41 percent in the piling weight required for stress only.

Piling schedules were presented for both conceptual designs (Attachment E) as required for stress only. It was noted that requirements for driving would be considerably heavier.

Verner noted that driving analyses had been initiated using McClelland Engineers' criteria of a quake of about 1 percent at the point when unplugged. This had given an optimistic result for a heavy wall pile and a Vulcan 060 hammer. Subsequent conversations with Lee Lowery at Texas A&M had indicated that he felt that the standard quake of 0.10 in. gave better results. An analysis using this quake at the point and a 2 in. wall thickness indicated marginal drivability to 190 ft and very doubtful drivability to the 230 ft required for the higher wave forces.

At this point Verner noted that if Lowery's recommendations were used, the full impact of the higher wave forces on the four-pile structure shown in Attachment C would be more than is indicated in Attachment D. He estimated that the structure would have to be 10 ft wider at the top and probably need 42 in. piles to handle the big hammer more easily, and to allow a larger area of steel without such high wall thicknesses.

Bodey guided the discussion onward to the drivability question. McCann presented a drivability analysis based on the wave equation procedure developed at Texas A&M. As noted by Cox and Johnston, this showed marginal drivability to the required penetration using a Vulcan 060 hammer on a 36 in. pile. It was noted that jetting would almost surely be required to get the pile to grade. Cox suggested that it might be a good idea to specify jetting at a particular stage in driving rather than waiting for refusal. He also noted a conversation on Friday, 21 May 1976 with Lee Lowery of Texas A&M in which Lowery commented that quake of 0.1 at the pile tip for all pile sizes seemed to give best results no matter if the pile was coring or plugged. Discussion followed concerning the effect this might have on the estimate of drivability. The conclusion was that this would definitely make drivability more questionable and heavier walls (2 inches) might be required even if jetted.

Bodey asked for alternatives and Johnston and Cox replied that drilling and grouting an insert would definitely get the pile to grade, but cost may be prohibitive. Verner raised the question of driving an insert pile. No one knew if that was reasonable. Cox suggested the possibility of a three-pile structure with skirts on the jacket as insurance. If piles were not driven to desired grades, the skirt piles could be driven. If the grade was reached with the main piles, it would not be necessary to drive the skirt piles. Verner noted that borings 2 and 3a showed considerably better driving conditions than boring 1. Alternates were summarized as (1) using even heavier wall than shown on the Crest preliminary designs, or (2) using extra skirts as insurance.

Bodey got back into the formal objectives of the meeting by asking if the skirt pile baseline configuration is the best design from the viewpoints of performance, schedule and cost. McCann replied that from Crest's point of view it would *not* be.

Bodey at this point wanted to split meeting to resolve two major issues: (1) effect of increased loads due to the change in wave theory, and (2) installation problems. Brill did not agree. His opinion was that the increased forces would have enough impact on possible installation problems that an estimate of the effects of the increases on base size and/or pile penetration should be settled before any discussion of installation started.

A short discussion followed in which it was concluded that the new pressures resulting from the stream function wave theory, with linear current with a reduction of 10 to 18 percent in moment due to the free surface effect, would fall somewhere between the forces generated by Glenn's wave theory with Const. Q and with Piggyback current coupling. Dean knew of a similar case in which the moment reduction on a monopile was 18 percent, but guessed that for a structure with horizontals the reduction would only be about 10 percent.

After lunch Verner presented calculations estimating the difference between the modified Stream Function and the basis of the conceptual design. These calculations were based on the overturning moment since it most directly affects the pile penetration. These calculations were based in part on Dean's results (Attachment B) and TERA's wave load sensitivity study (Attachment F) for 81 ft MLW. These calculations are shown below.

	Glenn-Piggyback	S.F.-10%	S.F.-18%	Glenn-Const. Q
Moment on mono-pile structure, 105 ft MLW (Attachment B)	17,260 'k	16,300 'k	14,800 'k	14,160 'k
Relative position	0.0	0.31	0.79	1.00
Moment on 4-pile structure, 81 ft MLW (Attachment C) at some relative position	128,700 'k	121,300	110,000	104,900

For same structure using same criteria as Crest used in the conceptual analysis, but TERA's Stokes 5th, the overturning moment was

$$M(\text{TERA}) = 103,100\ ^{\prime k}$$

$$M(\text{S.F.-10\%}) = \frac{121,300}{103,100} = 1.177\ M(\text{TERA})$$

$$M(\text{S.F.-18\%}) = 110,000/103,100 = 1.067\ M(\text{TERA})$$

Due to an error of about 10% in Crest's Stokes 5th under these conditions,

$$M(\text{TERA}) = 1.1\ M(\text{Crest})$$

$$\text{S.F.-10\%} = (1.177)(1.1) = 1.29\ M(\text{Crest})$$

$$\text{S.F.-18\%} = (1.067)(1.1) = 1.17\ M(\text{Crest})$$

A question was raised as to the origin of the 10 percent difference between Crest's and TERA's Stokes 5th solutions. The 10 percent was based on the 105 ft

site without current. To verify the same result for the 81 ft MLW site the following table was constructed by McCann and Verner showing the pressures at the crest used for conceptual design for 81 ft MLW where the only difference was the error in the Stokes 5th solution.

Pressure (psf)

Elevation	TERA	Crest	Percent Difference
Top	1012	921	+9.8
135	878	777	12.9
105	464	417	11.3
80	301	274	9.8
0	159	149	6.7

From this data, it was estimated that 10 percent would be about the resulting increase in overturning moment.

After it was noted that the increase from Crest's conceptual design pressures to pressures supplied by Dean would result in an increase of 17 to 29 percent in overturning moment, it was decided to tabulate the possible alternative in dealing with the resulting pile driving problems. These solution possibilities were noted as follows:

1. add six skirt piles
2. drilling and grouting (caving problem)
3. drive inserts
4. jetting and grout
5. larger hammer
6. add six skirt sleeves but use only if the main piles can't be driven or come back next summer if there is not sufficient time to drive the skirts
7. use 42 in. piles and a 300,000 ft-lb hammer such as the Vulcan 560

The next topic discussed was the question of possible fatigue problems on the East Coast. McCann noted that Crest intended to check two selected joints by analyzing the structure for about three or five different wave heights to find

the relation between joint stress and wave force. Bodey noted a paper by Kallaby and Price (#2609 OTC 1976) in which an allowable stress method was described which would allow a stress limit such as the API 20 ksi limit for the Gulf of Mexico. Verner discussed the method TERA would use which would assume a linear relation between wave force and stress to avoid additional analyses and require only wave force analyses for the different wave heights.

Procedures for earthquake analyses were discussed next. McCann said they would use the California procedure or the Navy procedure. He noted that although no earthquake zones are officially established for offshore sites at this time, API had recently published a map indicating the ACMR would be in Zone 1, but that it couldn't be official until January 1977.

The next topic for discussion was the project plan. Brill noted that he wanted documentation in the final plan showing, for example, the type of survey equipment that could be used. He noted the Navy's requirement for a single fabrication and construction contractor to eliminate contractural problems such as the installer going on day rate when fabrication is late. The project plan should include specifications for backup in evaluating bidders such as jacket location tolerance.

Consideration was given to eliminating design time from the project plan in order to move up the date of bid letting. McCann estimated five weeks would be required to get the structures through the computer the first time before turning on the drafting which would require about 8 weeks for a total of about 13 weeks. He noted that the skirt pile structures would take a few weeks less because drafting could begin earlier. It was noted that Crest's standard sheet size of 24 x 36 would simply be copied on the standard 27 x 40 military size.

An estimate was made of the time required to get preliminary sizes for TERA to get started checking. It was noted that for the skirt pile preliminary sizes are available immediately. For a three- or four-pile structure, McCann estimated they might possibly get structures through the computer in 4-1/2 weeks

if Dean can respond in one week. McCann noted that the addition of skirt sleeves to the structures would not affect timing very much.

TERA noted that after receiving preliminary sizes from Crest, their feedback to Crest would begin after a lapse of about two weeks, but final checking would take at least the full seven weeks (5th through 11th week of Design Phase) being contemplated, provided three engineers are available and final reports, etc. are left until later. Erchul noted that the contract must be awarded at the beginning of December.

Brill inquired as to how much time should be allowed for bidder response. McCann said he had used the Government recommendation of five weeks for bidders and two weeks for the Government evaluation. Ling noted that the two week Government evaluation assumed that bidders don't spot any problems in contract documents requiring amendments.

Erchul emphasized the need to be at least semi-operational by the end of the summer. He felt that this would require sites 3 and 4 to be installed, but said he would check with NAVAIR. McCann noted that this would be difficult because it would require scheduling both large structures for the same barge which would cause overhang and increase insurance costs.

Wednesday, 26 May 1976

Present:

Navy	Crest	TERA
Erchul	Johnston	Cox
Bodey	McCann	Verner
Brill		Vines
Ling		
Raecke		
Masso		

Bodey began the meeting by recapping Tuesday's meeting.

A brief discussion of drivability followed in response to a question by Erchul. It was noted the driving resistance of a larger (42 in.) pile would be about the same as a smaller (36 in.) pile at design penetration since the overturning moments would be about the same and hence the required static capacity would be about the same. Lowell Johnston expressed the opinion that it is actually easier to drive a bigger pile to a shallower depth than a smaller pile to a deeper depth.

Shun Ling began a discussion on the time schedule for the project plan. By eliminating two weeks in the design phase and four weeks in the review phase, it was estimated that six weeks could be saved to allow the contract to be awarded sometime in December.

Crest had had a wave equation analysis run for the Vulcan 560 hammer (300 ft-kips). The results were presented as follows in tons:

	200 ft Penetration		250 ft Penetration	
Quake at Point	200 BPF	300 BPF	200 BPF	300 BPF
For 36" x 2" Pile				
0.025	2300	2450	2250	2400
0.1	2000	2070	1950	2050
0.3	1550	1640	1550	1632
For 42" x 1.75" Pile				
0.025	2350	2500	2300	2500
0.1	2000	2125		2125
0.3	1575	1700		1675

It was concluded that the 560 would stand a very good chance of driving the piles with jetting and might be able to drive without jetting. It was noted that Santa Fe, Raymond, B&R, and McDermott all have hammers of similar energy ratings to the Vulcan 560. Cox suggested he should talk with Bill Drawe of Burmah Oil and Gas Company who has had some experience with the Vulcan 560 on 48 in. piles in sand.

In considering whether to increase pile size from 36 in. to 42 in., Verner suggested it might mean the difference between 3 add-ons and 2 add-ons. Johnston estimated that for a 42 in. pile, the first section could be 150 ft long and still be within the handling capabilities of the anticipated derrick barge boom length. This might get to its 30 ft of penetration without driving. The second add-on could probably be 90 ft with a 42 in. pile, leaving about 20 ft or so to get to 190 ft of penetration which would probably be required for the higher loads. It was estimated that this schedule could probably be stretched to accommodate the deeper water structures. From these calculations, it was evident that the 36 in. pile would probably require an extra add-on since its length would be greater and the allowable add-on length would be less.

Crest recommended the use of the 42 in. pile instead of the 36 in. since it would probably save a total of about 6 days installation time, and more easily accommodate the anticipated higher loads and large hammer size.

Bodey guided the discussion on to the question of the return interval. Johnston noted an economic study by Pete Marshall in Ocean Engineer(?) which indicated a 40 to 80 year return period (design life?) to be most economical. He also noted that the British Government is specifying a 50 year return period for manned platforms in the North Sea. Bodey said the Navy would go ahead with the 50 year return period, but may back off if problems are encountered or at least evaluate the risk.

Ling suggested an evaluation of the cost impact of 20 percent increase in loads and a 42 in. pile. It was noted that transportation cost would be the same

and that material costs would be only in the piling and jacket legs. The increase in leg weight was estimated to be 60 tons for all four legs or $120,000. The piling was estimated to increase by 20 percent for a total of 220 tons or $200,000. The total increase in material cost was estimated to be $340,000. (Note: the 6 day installation saving for 42 in. piles would just about offset this.)

Erchul inquired if the weather time is based on data or arbitrarily selected. McCann replied that he used Glenn's data for June, July, and August:

57.7%	under 4 ft	=	operational
37%		=	limited operational
5%	over 10 ft	=	shut down

Brill noted that he would like wave heights for starting different item operations. He noted the need to define weather days for contractural considerations. Lowell Johnston suggested the rule of three's for flat bottomed barges like the Lindsay, and the rule of four's for barges with ship hull:

Rule of three's	Rule of four's
3 ft operational	4 ft operational
6 ft marginal	8 ft marginal
9 ft shut down	12 ft shut down

Cox reported the results of a phone call by Cox and Brill to Bill Drawe of Burmah Oil and Gas Company. Drawe noted that McDermott has a Vulcan 3100 hammer and Brown and Root has a Vulcan 560 hammer, both of which are rated at 300 ft-kips of energy. Drawe described his experience with a structure where the soils consisted of 100 ft of sand at the surface, underlain by clay which was underlain by dense sand. He had 48 in. x 1.5 in. piles with 50-55 ft add-ons and drove to 325 ft penetration without having to jet using the driving shoe shown below to prevent the formation of a soil plug.

He noted that the Vulcan 560 hammer got half the blows per foot of the Vulcan 060 and that most driving with the 560 was at 110 to 120 bpf. He said he didn't like the idea of a 1:6 batter with a 42 in. pile. He noted that he has a design for a structure with 42 in. piles using the 300 ft-kip hammers.

The question of who would be likely to bid on the project was raised by Brill. He asked for the opinion of Cox and Johnston on the following contractors:

Gulf Coast	Johnston	Cox
Brown and Root	1/2 Bid	
R.I.	1 Bid	Est. 5 Bids
McDermott	1/2 Bid	
Teledyne	1 Bid	
Santa Fe	0	
Williams	0	
East Coast		
Tidewater	----	
Klein	1/2 (want to fab)	Est. 3 Bids
Marathon	1/2 (want to fab)	
Hewlet	----	

Miscellaneous discussions consumed the remainder of the meeting.

EAST COAST ACMR OCEAN TOWERS

SYSTEM ANALYSIS REVIEW

PHASE A, CONTRACT N 62477-76-C-0179

WASHINGTON D.C. MAY 24, 25, 26, 1976

ATTACHMENT A

SYSTEM ANALYSIS

OBJECTIVE

PROVIDE BASIS FOR RESOLUTION OF CRITICAL ISSUES AS PREREQUISITE TO PHASE B, DESIGN ENGINEERING

SCOPE

Ⓐ ENVIRONMENTAL DESIGN CRITERIA
- WAVE THEORY
- WAVE & CURRENT COUPLING
- WAVE SPECTRUM FOR FATIGUE

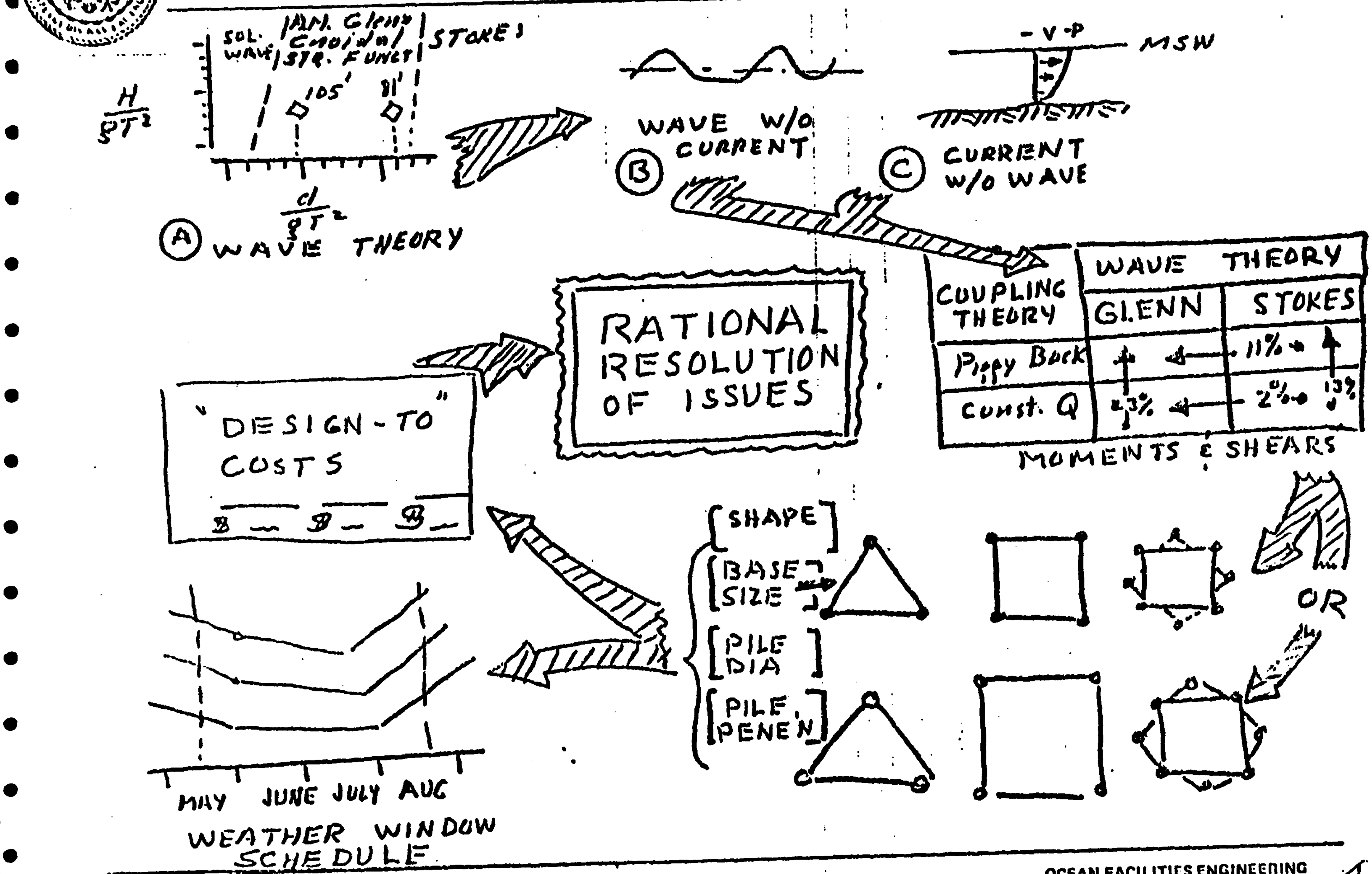
Ⓑ STRUCTURAL CONCEPT VALIDATION

© PROJECT COST/SCHEDULE PLAN

SPECIFIC QUESTIONS

1. CAN GLENN'S "PROPRIETARY" WAVE THEORY BE RATIONALLY USED AT 81' & 105' SITES?
2. CAN GLENN'S CURRENT-COUPLING THEORY BE RATIONALLY USED?
3. IS BASELINE SKIRT-PILE CONFIG'N BEST FOR PERFORMANCE/SCHEDULE/COSTS?
4. CAN PROGRAM COST & SCHEDULE BE MET?
5. IS FATIGUE A BIGGER DESIGN DRIVER OFF HATTERAS --- THAN THE GULF? HOW ESTABLISHED?

WHY WE'RE HERE?



AGENDA

MONDAY, 24 MAY

8:30 - 8:45	ASSEMBLY & INTRODUCTION [CDR ERCHUL]
8:45 - 9:00	BACKGROUND & MEETING PLAN [BODEY]
9:00 - 10:00	ENVIRONMENTAL REPORT [McCANN]
10:00 - 11:00	ENVIRONMENTAL REPORT [VERNER]
11:00 - 11:30	DISCUSSIONS
11:30 - 1:00	LUNCHBREAK
1:00 - ON	WORK SHOP

- IDENTIFY ISSUES
- PREPARE INFO TO RESOLVE
- PRESENT INFO & HOLD DISCUSSIONS
- GOV'T POSITION
- FOLLOW-UP ACTION ASSIGNMENTS
- ADJOURN ~ OR NEXT SUBJECT

AGENDA [CONT'D]

TUESDAY, 25 MAY

8:30 - 8:45	ASSEMBLY & RECAP [BODEY]
8:45 - 11:00	STRUCTURAL REPORT [McCANN]
11:00 - 11:30	STRUCTURAL REPORT [VERNER]
11:30 - 1:00	LUNCHBREAK
1:00 - 2:00	DISCUSSION OF ABOVE
2:00 - 3:00	GOV'T. POSITION, FOLLOW-UP ACTION ASSIGNM'TS
3:00 - 3:30	• CREST PRENTATION ON ØB FATIGUE ANALYSIS PLAN [McCANN]
3:30 - ON	• DISCUSSIONS ON FATIGUE ANALYSIS • ACTION ASSIGNMENTS • ADJOURN, OR NEXT SUBJECT

AGENDA [CONT'D]

WEDNESDAY 26 MAY

8:30-8:45	ASSEMBLY & RECAP [BODEY]
8:45-9:30	PROJECT PLAN TO: [McCANN] • MEET "DESIGN-TO" COSTS, • INSTALLATION IN SUMMER '77, • WORK AROUND PLAN(s).
9:30-10:30	DISCUSSIONS
10:30-11:30	GOV'T POSITION & ACTION ASSIGN'TS
11:30-1:00	LUNCHBREAK
1:00-ON	FINAL DISCUSSION, ALL SUBJECTS RECAP GOV'T POS'N RECAP ACTION ASSIGN'TS ADJOURN THE REVIEW

PRELIMINA

SIN - E PILE (D=3')

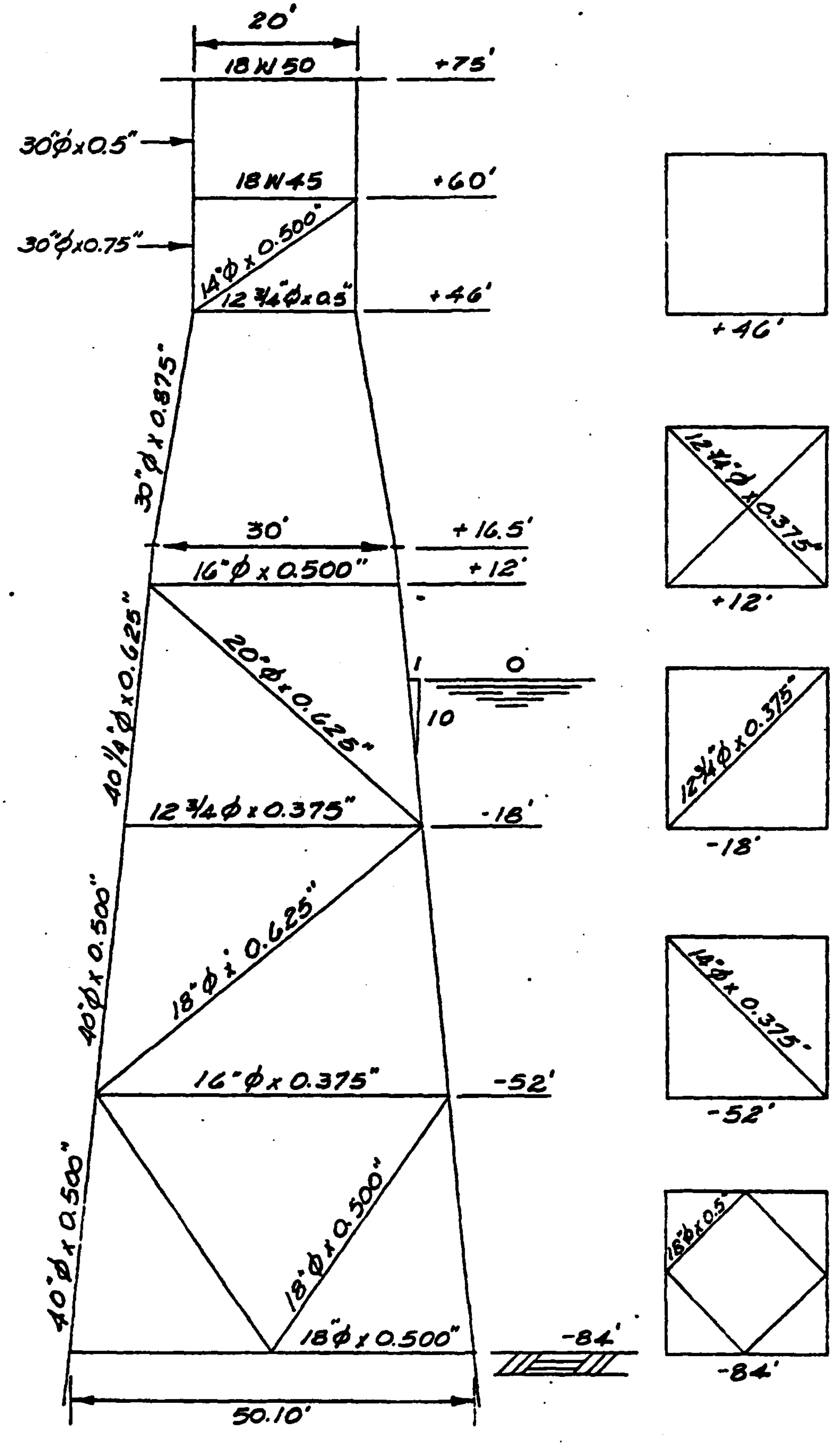
$C_D = 0.74$

$C_m = 1.34$

PARAMETER		THEORY / CURRENT COMBINATION				
SITE		GLENN "PIGGY BACK"	GLENN "CONSTANT Q"	STOKES V "PIGGY BACK"	STOKES V "CONSTANT Q"	STREAM FN III
I_1=112.7' H=61.3' T=13.6.5	RANGE OF DISP. VALUE			4.77 ft	8.15 ft	0.06
	$(F_D)_{max}$ (KIPS)	165.93	34.27	180.75	150.19	168.4
	$(M_D)_{max}$ (K-FT)	17,260	14,160	18,610	15,540	18,066
	$(F_I)_{max}$ (K)	13.4	13.3	15.4	15.6	14.6
	$(M_I)_{max}$ (K-FT)	1008	935	1167	1213	1,204
h=89.5' H=60.3' T=13.6'	RANGE OF DISP. VALUE			11.58 ft	5.88	1.62
	$(F_D)_{max}$ (K)		162.8	218.5	178.5	200.8
	$(M_D)_{max}$ (K-FT)		15,300.	19,680	16,930	19,820
	$(F_I)_{max}$ (K)		13.0	15.1	15.9	14.0
	$(M_I)_{max}$ (K-FT)		767	1161	1083	1040

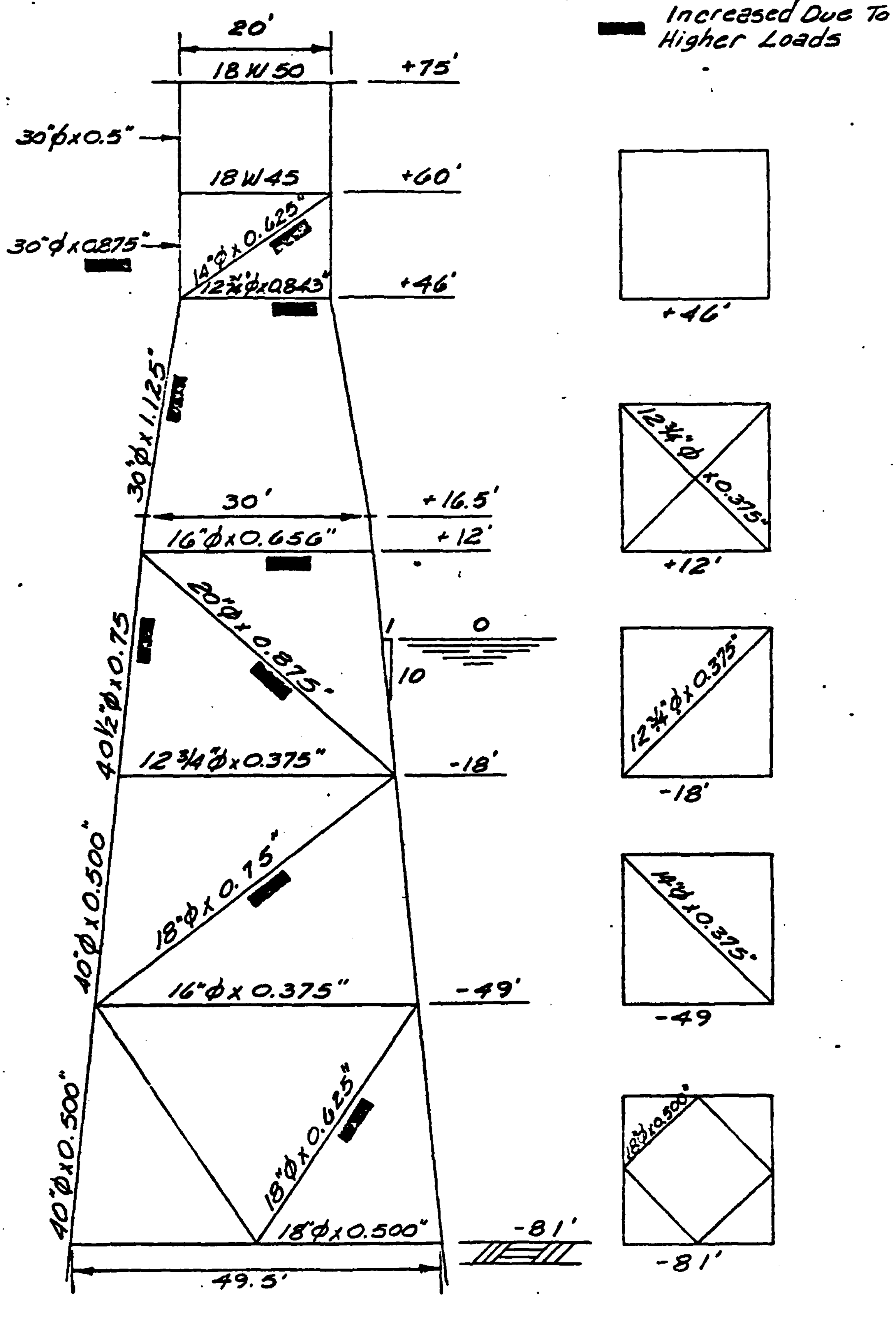
RGS 5/?

ATTACHMEN



FOUR PILE CONCEPT, 84' MLW, NAVY
CRITERIA, STOKES 5TH WAVE, CONST. ℘

ATTACHMENT C



FOUR PILE CONCEPT, 81' MLW, GLENN CRITERIA, GLEN WAVE, APPROX. PIGGYBACK

ATTACHMENT D

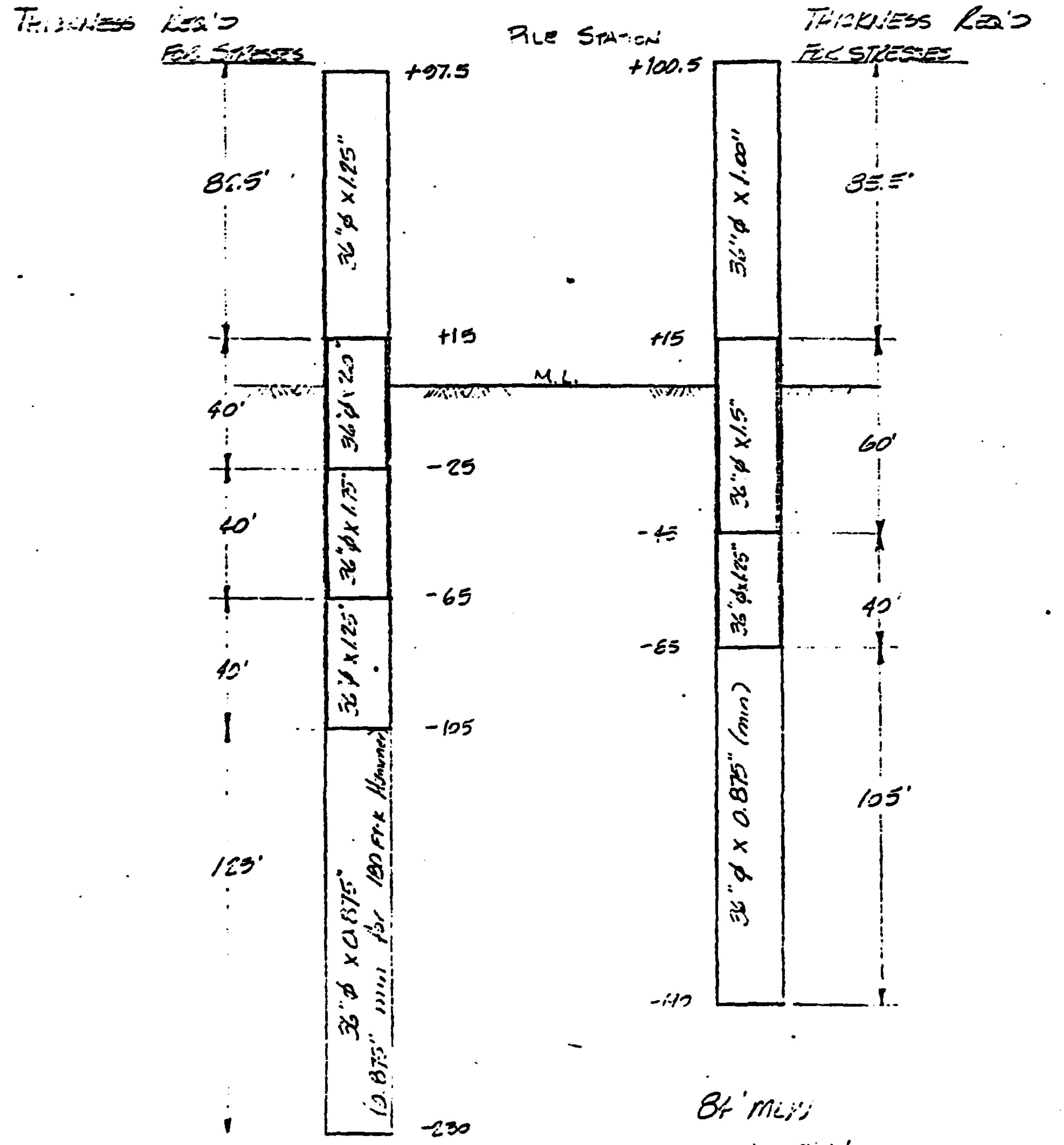
TERA, INC.
OCEAN ENGINEERING

SHEET NO. ______ OF ______

CLIENT ______ JOB NO. ______

SUBJECT ______

COMPUTER ______ DRAWING NO. ______ DATE ______



84' MLW

STOKES 5TH WAVE

"CONSTANT D" ...

ATTACHMENT E

OCEAN ENGINEERING

SHEET NO. 1 OF ____

CLIENT [illegible] JOB NO. 73-[illegible]

SUBJECT [illegible]

COMPUTER [illegible] DRAWING NO. ____ DATE [illegible] 76

WAVELOADS – 4 Pile Structure

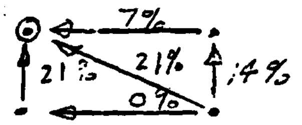
	270° F	270° M	315° F	315° M
1. 84' – NAVY CRITERIA STOKES 5 - CONST. Q	1215	98,340	1264	103,100
2. 81' – GLENN CRITERIA STOK 5 – CONST Q	1231	94,910	1237	99,960
3. 81' – GLENN CRITERIA STOK5 – PIGGY BACK	1401	109,370	1463	115,280
4) 81' – GLENN CRITERIA GLENN WAVE - CONST. Q	1234	99,350	1284	104,890
5) 81' – GLENN CRITERIA GLENN WAVE – APPROX PIGGYBACK	1491	121,780	1560	123,705

FORCE (315°)

GLENN STOKES

PIGGYBACK

CONST Q

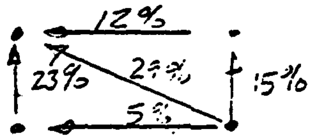


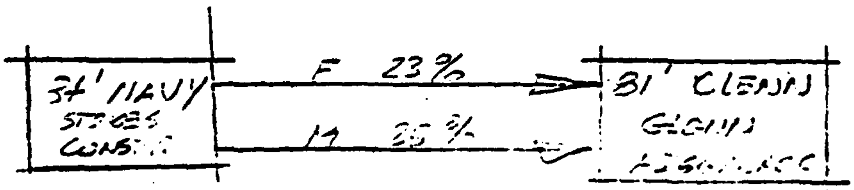
MOMENT (315%)

GLENN STOKES

PIGGYBACK

CONST. Q.





ATTACHMEN

OCEAN ENGINEERING

SHEET NO. 1 OF ____

CLIENT [illegible] JOB NO. [illegible]

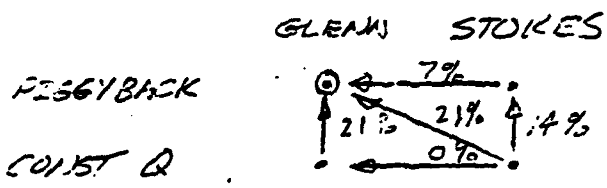
SUBJECT [illegible]

COMPUTER [illegible] DRAWING NO. ____ DATE [illegible] 76

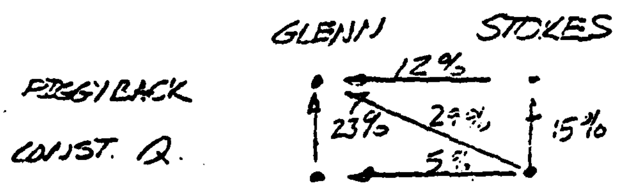
WAVELOADS – 4 Pile Station

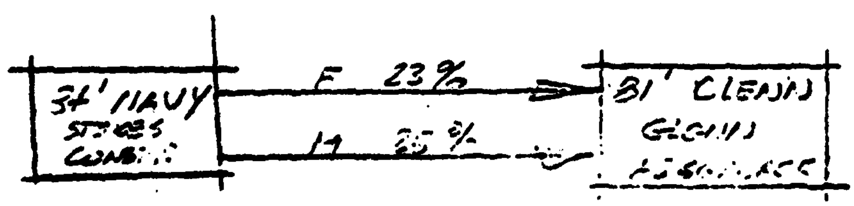
	270° F	270° M	315° F	315° M
1. 84' – NAVY CRITERIA STOKES 5 - CONST. Q	1215	98,340	1264	103,100
2. 81' – GLENN CRITERIA STOK 5 – CONST Q	1231	94,910	1254	99,960
3. 81' – GLENN CRITERIA STOK5 – PIGGY BACK	1401	109,370	1463	115,280
4) 81' – GLENN CRITERIA GLENN WAVE - CONST. Q	1234	99,350	1284	104,893
5.) 81' – Glenn Criteria GLENN WAVE – APPROX PIGGYBACK	1491	121,780	1560	123,705

FORCE (315°)



MOMENT (315%)





ATTACHMENT

APPENDIX B

REPORT BY DEAN

APPENDIX C

INDEX TO CALCULATIONS

LIST OF CALCULATIONS

Notebook #1 - Environmental Study

A. Data supplied to R. G. Dean
   Wave 1.4 runs (5) 4 on 17 May 76, 1 on 9 April 76

B. Data supplied by J. W. McCann 7 April 76
   Wave profiles - Stokes 5th w/no current and Glenn's w/Const Q

C. Check of Stokes 5th solutions (hand calculations) 4 May 76

D. Data supplied to J. W. McCann 19 May 76
   TERA's Stokes 5th Solution

E. Study of effect of criteria on 81 ft MLW structure
   2 - Wave 1.4 runs
   3 - Waveld runs
   Hand calculations summarizing data 18 May 76

+ Dean's Data

LIST OF CALCULATIONS (Cont'd)

Notebook #2 - Structural Concept Analysis - General

A. Comparison of p-y data
   - 5 Pygen runs (parameter study)
   - 4 pages of notes and graphs showing comparison 11 May 76

B. Crest's Wave Criteria - Confirmation of Agreement
   - 2 - Wave 1.4 runs
   - 1 - Waveload run
   - 1 page of notes on member sizes 10 May 76

C. Wind Loads and Dead Loads used in "Stress"
   - 3 pages of notes and calculations 20 May 76

D. Pile Drivability Assumption
   - 2 pages of notes and calculations 3 May 76

E. Design Axial Capacity Curves
   - 2 pages of notes and calculations 3 May 76

F. Rough Size Structure vs. Pile Size
   - 4 pages notes and calculations 5 May 76

G. Structural Model
   1. Structure numbering 84 ft MLW - 12 pages of notes 10 May 76
   2. Coordinates for equivalent pile member
      1 page notes and call 11 May 76
   3. Original of member check table - 3 pages
   4. Maximum wall thicknesses - 1 page 20 May 1976

LIST OF CALCULATIONS (Cont'd)

Notebook #3 - 84 ft MLW 4-Pile Concept

A. Structural Model
  1. Structure numbering
     6 pages of drawings — 10 May 76
  2. Equivalent member for pile
     2 - BMCONL Runs
     3 pages of notes and calculations — 12 May 76
  3. Revised Moment Restrain
     1 page summary and calculations — 14 May 76
  4. First Waveld Iteration
     2 - Waveld runs
  5. Structural check
     1 - Stress Run
     Check of initial member sizes - 4 pages w/Rev's
     Revised member sizes (Drawing) — 20 May 76
     Weight take-off - 2 pages — 18 May 76
  6. Second Waveld Iteration
     1 - Waveld run w/comparison calculations — 17 May 76
     Jacket weight takeoff - 3 pages — 17 May 76
  7. Pile Design
     3 - BMCONL runs
     Static pile design - 2 pages notes, calculations and drawings — 18 May 76

STRESS Wave ?

LIST OF CALCULATIONS (Cont'd)

Notebook #4 - 81 ft MLW 4-Pile Concept

A. Structural Model
- 6 pages drawings — 18 May 76

B. Equivalent Pile Member
- 2 pages notes and calculations — 19 May 76
- 1 - BMCONL run — 19 May 76

C. First Waveld Iteration
- 1 - Polynomial run — 19 May 76
- 1 - Waveld run — 19 May 76
- Comparison of Glenn's Wave w/Piggyback Coupling and Glenn's Wave w/Direct Superposition for 105 ft MLW 3 pages notes, calculations, and drawings — 14-19 May 76

D. Structural Check
- 1 - STRESS run
- Check of initial member sizes 4 pages w/Revs
- Weight takeoff 2 pages — 22 May 76

E. Pile Design
- 1 - BMCONL run
- Pile Design 2 pages notes, calculations, and drawings — 20 May 76

APPENDIX D

MEMORANDA

The following is a list, by date, of all memoranda written or received by this office relating to the execution of Phase A, Systems Analyses of Contract N62477-75-C-0112.

Personae	Date	Subject of Memorandum
NAVFAC to TERA	04 Mar 76	Notice to Proceed
EAV to NAVFAC	30 Mar 76	Notice to Proceed Signed
EAV to CB, JMc, DR	02 Apr 76	Phone Con re Clarification of Parameters in Crest's Environmental Study
EAV to CB, SL	06 Apr 76	Phone Con Re Course of Environmental Studies
EAV to JMc	08 Apr 76	Phone Con Re Current Coupling Technique; Stokes 5th Solutions
EAV to JMc	09 Apr 76	Phone Con Re Dean's Suggestion for Obtaining Riding Current
EAV to RGD	09 Apr 76	Phone Con Re TERA's Method of Current Coupling
EAV to RGD	13 Apr 76	Phone Con Re Current Coupling Techniques
EAV to JMc	13 Apr 76	Phone Con Re Riding Current and Combining Velocities
JMc to EAV	13 Apr 76	Phone Con Re Simplified Method for Obtaining Riding Current
DR to EAV, BC	19 Apr 76	Phone Con Re REQ's for Crest's 30% Submittal
EAV to SL	27 Apr 76	Phone Con Re Status of Quads; Dean's Work; Phase B Proposal (P0003)
CB to EAV	28 Apr 76	Phone Con Re P0003; Glenn's Data; Meeting
EAV to CB, SL	30 Apr 76	Phone Con Re Pile Drivability Report; Dean's Work

END
FILMED
3-86
DTIC